In Partial Fulfillment of Consent Order
Requirements
CERCLA Docket No. 87-1.

GROUND WATER MIGRATION MANAGEMENT FEASIBILITY STUDY

SHERIDAN DISPOSAL SERVICES SITE
WALLER COUNTY, TEXAS

014612

Prepared for:

The Sheridan Site Committee

July 25, 1989

W.O. #91-21

Prepared by:

ERM-SOUTHWEST, INC.
16000 Memorial Drive, Suite 200
Houston, Texas 77079
(713) 496-9600

L880

518

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

The Sheridan Site Committee has investigated the Sheridan Disposal Services site near Hempstead, Texas. The ultimate objective of the Ground Water Feasibility Study is to provide a basis for selecting a cost effective remedial alternative that is protective of human health and the environment. The alternatives evaluated are designed to meet this and other remedial objectives as well as attain Federal and State requirements that are applicable or relevant and appropriate (ARARs). The investigation and evaluation of this site has been divided into a source control effort and a ground water migration management effort. The ground water migration management effort has resulted in a Ground Water Remedial Investigation and this document, the Ground Water Feasibility Study.

The facility currently occupies approximately 110 acres and includes a 42 acre evaporation system, a main pond whose surface area varies from twelve to fifteen acres (depending on water level), and a seventeen acre dike area around the main pond. Some inoperable equipment and a group of nine treatment and storage tanks are located on the east side of the main pond on the levee. Remaining acreage consists of borrow ditches excavated for the dikes and other "buffer zone" areas inside the perimeter fence.

Following site characterization, ground water migration control technologies were screened from possible general response actions and assembled into alternatives. These alternatives underwent careful design analysis, including sufficient design development to enable a detailed cost estimate to be prepared. The following four alternatives survived the screening process and were carried forward into the detailed analysis phase:

- o Take no action at the site.
- o Take limited action, including ground water monitoring.
- o Construct a slurry wall between the site and the Brazos River; recover and treat ground water.
- o Construct a line of wells between the site and the Brazos River; recover and treat ground water.

In the detailed alternative analysis phase, each of these alternatives was ranked based on relative compliance with ARARs; reduction of toxicity, mobility or volume; short-term effectiveness; long-term effectiveness and permanence; implementability; cost; and overall protection of human health and the environment. Based on this detailed evaluation it will be possible to select a cost-effective remedial alternative consistent with the objectives outlined above.

GROUND WATER MIGRATION MANAGEMENT
FEASIBILITY STUDY

SHERIDAN DISPOSAL SERVICES SITE
WALLER COUNTY, TEXAS

1 - INTRODUCTION

1.1 Purpose and Scope

The purpose of this Ground Water Migration Management Feasibility Study (GWFS) is to present the process and results of the development of the ground water remedial alternatives for the Sheridan Disposal Services (SDS) site. This GWFS is based on the information and data presented in the Ground Water Migration Management Remedial Investigation (GWRI) and the Source Control Remedial Investigation (SCRI) by the Sheridan Site Committee (SSC) and the November 1988 Baseline Risk Assessment (RA). The GWRI characterized the ground water and defined the hydrologic character of the site. The RA addressed the necessity of remediation by evaluating risks. By addressing the risks identified in the RA, the GWFS alternatives will be protective of human health and the environment.

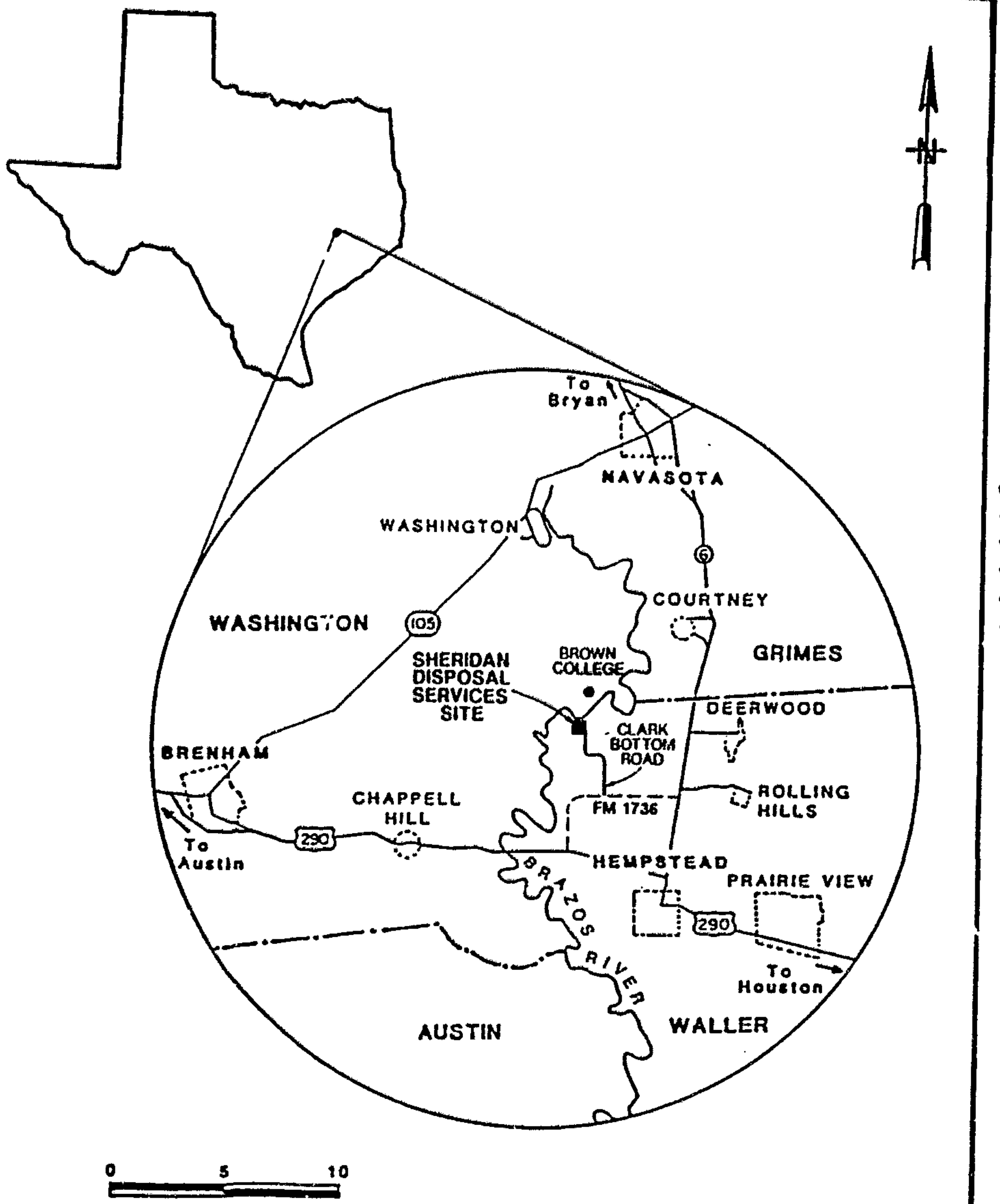
014617

The GWFS identifies and analyzes ground water migration control alternatives that are consistent with the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA), as amended by the Superfund Amendments and Reauthorization Act (SARA), and the National Oil and Hazardous Substances Pollution Contingency Plan (NCP), and which effectively mitigate and minimize threats to, and provide adequate protection of, public health and welfare and the environment at the site.

1.2 Site Description

1.2.1 Geographical Location

The SDS site is located in Waller County, Texas, approximately nine miles north-northwest of the City of Hempstead, Texas and two miles northwest of the intersection of Clark Bottom Road and Farm Road 1736 (Fig. 1-1). The property is bounded on the east, south and west sides by farm and ranch lands, and on the north by the Brazos River. The site lies within the Gulf Coastal Plain Physiographic Province and is transitionally positioned between the Post Oak Savannah and Blackland Prairie Natural Regions of Texas.



SOURCE: REI, MARCH 1986

014618

ERM-Southwest, inc.
NEW ORLEANS, LOUISIANA HOUSTON, TEXAS

DATE 2/27/89	W.O.NO. 9121A004

FIGURE 1-1

SITE LOCATION MAP

Sheridan Disposal Services
Hempstead, Texas

R235

1.2.2 Site Geology

The SDS site lies on the Brazos River Alluvium along the southern bank of the Brazos River. At this location the alluvium is approximately 55 feet thick and is composed of recent river bottom and overbank deposits. Sediment types range from clay to gravel. Two divisions of the alluvium have been designated in the SCRI: stratum A and stratum B. Stratum A extends from the surface to as much as 40 feet in depth, although it is generally 25-30 feet thick. It is composed of reddish brown interbedded sands, silts and clays. These vary in their lateral extent and composition depending on the distance from the river. For example, sediments near the river bank tend to be sandier with lenses of clay and have small lateral extents, whereas further south, away from the river, these sediments are much clayier. Correlations of individual beds within stratum A are difficult to make across the site due to the variable nature of stratum A.

Stratum B is the lower portion of the alluvium and can be distinguished from stratum A where sand becomes prevalent and continuous with depth. Stratum B is a typical alluvial fining-upward sequence for a meandering stream. The upper portion generally begins as a fine sand or silty sand that continuously grades downward into coarse sand or pebbles and sometimes gravel at or near its base. These sediments are reddish tan to brown and become grey with increasing grain size. They are poorly-to well-sorted and are composed primarily of quartz and chert grains with occasional shell fragments. This sequence is found all over the site, even though its thickness may vary. Portions or all of stratum B are saturated and under unconfined conditions. This forms the uppermost aquifer (referred to as the unconfined or alluvial aquifer) under the SDS site.

Underneath the alluvium is the Fleming Formation. At the site the Fleming has been subdivided into three units: stratum C, stratum D and stratum E. Stratum C is generally a dense, hard olive green to grey clay which is laterally continuous under the site, usually found between 55 and 60 feet below grade. It often contains layers of clayey silt or clayey sand within it. These coarser-grained layers are up to two feet thick. South of Clark Lake at MW-40, approximately 1700 feet south of the southern edge of the main pond, stratum C is primarily a clayey or sandy silt.

The thickness of stratum C is quite variable, ranging from twelve feet at MW-40 to 37 feet at MW-14. Based on available data from boreholes drilled through October 1987, the average thickness of stratum C under the site is approximately 27 feet.

Beneath this clay is stratum D, a sandy transmissive zone that is saturated and under confined conditions. It is the second aquifer underlying the site. Depending on location, stratum D is found between 65 and 95 feet below grade. Its thickness varies from 9.5 feet to 28 feet, with an average value of 16.5. Stratum D is a reddish or brown to tan, fine to medium-grained sand that is silty or clayey in zones. In some areas this sand is very well cemented with calcareous cement, forming a hard sandstone layer near the top of the stratum.

Stratum E is beneath stratum D and is another very hard and stiff clay layer. It is grey with olive mottling and iron staining and has a similar appearance to stratum C. The lateral extent and thickness of stratum E under the site is unknown.

1.2.3 Facility Description

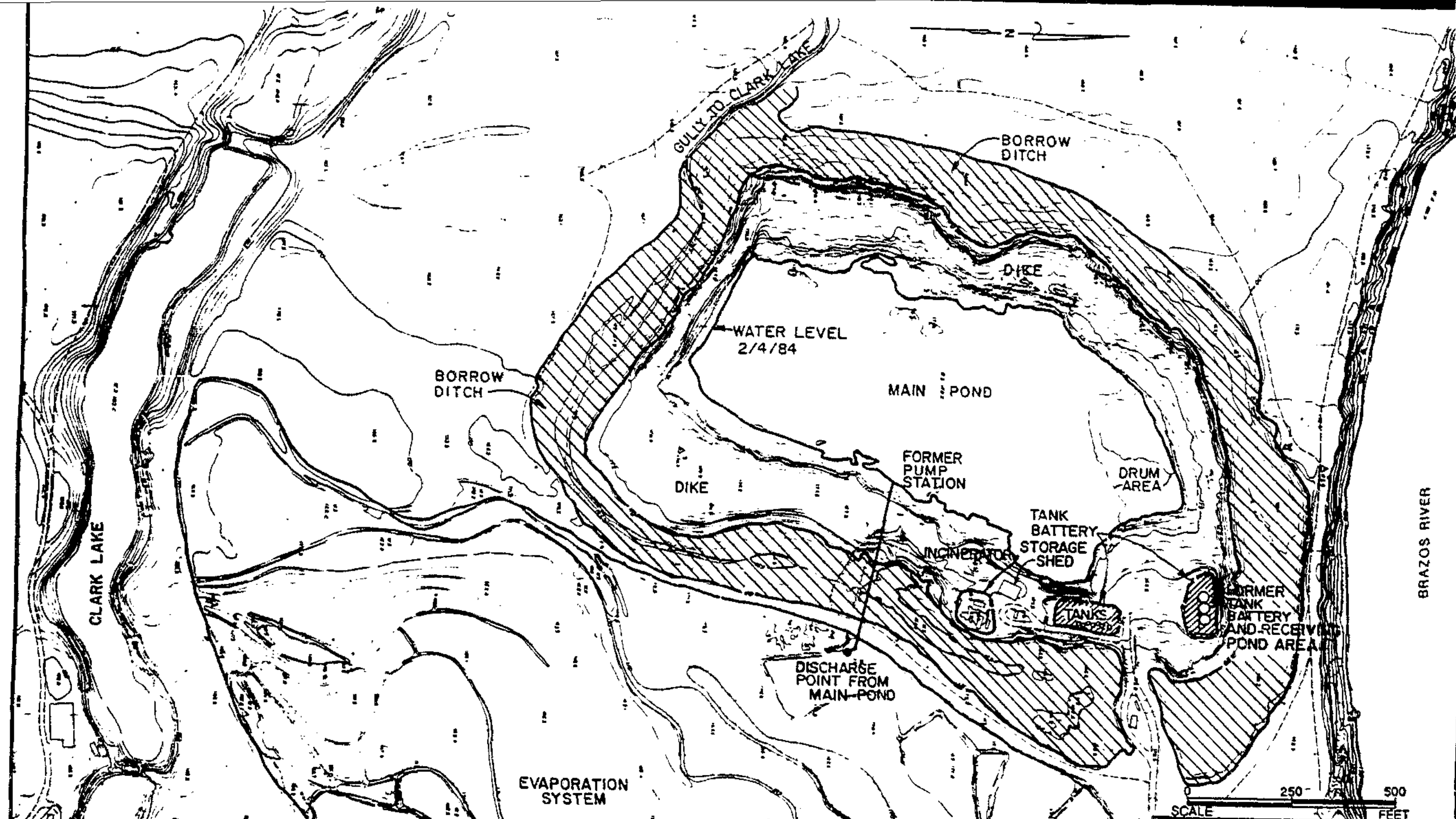
The facility currently occupies approximately 110 acres, and includes a 42 acre evaporation system, a twelve to fifteen acre main pond and a seventeen acre dike area around the main pond (Figure 1-2). Some inoperable equipment and a group of nine treatment and storage tanks are located on the east side of the main pond on the levee. Remaining acreage consists of borrow ditches excavated for the dikes and other "buffer zone" areas inside the perimeter fence.

The main pond was located in a naturally occurring, low-lying area that was gradually expanded to about 22 acres utilizing a system of dikes. The main pond was used as a surface impoundment for material disposal and for open pit burning. Partial closure activity reduced the size of the main pond to approximately fifteen acres.

Water that accumulated in the main pond due to precipitation was pumped into the evaporation system and allowed to evaporate. The tanks were used for the separation and treatment of incoming liquid waste.

1.2.4 Chronological History of Site Management, Use and Modifications

The SDS site, owned and operated by Mr. Duane Sheridan of Hempstead, Texas, began accepting industrial wastes for disposal in the late 1950s. These wastes were disposed of by open pit burning and surface impoundment of ash residue in a naturally low-lying area of Mr. Sheridan's property. As the volume of material accepted at the site increased, a levee composed of native soils



ERM-Southwest, inc.

FIGURE 1-2

SITE FACILITIES MAP

Sheridan Disposal Service

Hempstead, Texas

W.O. NO. 9121B007 | 2/27/89

014621

and combustion residuals from waste burning was constructed around the pit area to form the main pond (sometime in late 1963). These site management practices and facilities were used through 1971.

A group of storage and treatment tanks were constructed beginning in September 1971 in response to an order from the Texas Water Quality Board (TWQB), a predecessor to the Texas Water Commission (TWC). These tanks were used for steam treating oil-water emulsions. Separated oils were used as fuel for a system of ground flares that was installed in 1972.

A smaller pond (approximately 400,000 gallons) was constructed in the northwest corner of the main pond dike (Figure 1-2). It was used to receive incoming materials. From there the waste was generally pumped into the steam treatment system for emulsion treatment. Any recovered oils were either sold or used for ground flares or boiler feed to generate steam. Leftover residues were disposed of in the main pond. Liquid wastes from the smaller pond were discharged directly into the main pond when the steam emulsion system was not working or was over-loaded.

In November of 1972 a fire destroyed the oil burner (incinerator) system and the surface of the main pond was ignited, burning off the surface layer of oil. In 1969 and 1973 severe rainfall caused apparent overflows. The height of the dike was increased in 1975 to mitigate this occurrence.

During 1974 and 1975, several trial burns of new incinerators were performed by SDS. Permit approval was granted to SDS by the Texas Air Control Board (TACB) for a liquid waste burner (incinerator) that was designed and built by Mr. Sheridan. The incinerator was in use until June 1978 when a fire destroyed portions of the system.

In order to take care of the continuing problem of accumulated pond stormwater, a new facility -- the evaporation system -- was built in 1976 adjacent to the main pond. This 42 acre impoundment received wastewater from the main pond into a series of small cells where it was allowed to evaporate.

SDS began closing the main pond with dike and other materials in October 1978. An initial closure plan was agreed to by SDS and the TWQB in 1979. This plan called for initial closure of the main pond, pumping of accumulated stormwater from the pond into the evaporation system, and maintenance of the pond dike. Pond water was transferred to the evaporation system and approximately seven acres of the main pond, corresponding to the receiving basin, was covered with fill material.

A final closure plan was submitted to the Texas Department of Water Resources (TDWR, now the TWC), by SDS on December 2, 1983. It was rejected in January 1984, at which time the TDWR determined that SDS did not have the expertise or resources to properly close the site. At that time the TWC contacted certain companies, whose waste may have been disposed of at the SDS site, to request assistance in the site closure. The SSC was formed by certain of those companies in response to that request and has since worked with the TWC and the EPA to collect and analyze information necessary to evaluate appropriate closure alternatives.

The SDS site was proposed for inclusion on the National Priorities List pursuant to Section 105 of CERCLA on June 10, 1986.

1.3 Source Material Description

The major sources of organic and inorganic chemical constituents are the main pond water and sludges. Manifest descriptions of some of the materials received for disposal at the site are listed in Table 1-1. Summaries of organic compounds and metals found in all sources are presented in Table 1-2 (footnotes and references in the table relate to the Baseline Risk Assessment).

Main Pond

The current surface area of the main pond varies between twelve and fifteen acres. The depth of the accumulated rainwater varies between one and six feet during periods of accumulation and removal.

The main pond contents are stratified into a partial surface oil and emulsion layer, an aqueous phase and a heavy sludge layer. The surface oil layer (less than two inches in thickness) currently covers less than fifteen percent of the pond surface and varies depending on wind conditions. At the present time, the majority of the oil layer has been removed from the main pond and the main pond water has been evaporated in the evaporation system in accordance with an Administrative Order issued by the EPA.

Based on results from the analysis of fifteen samples collected in June 1987, the main pond sludges vary in thickness from about six inches to about 24 inches, with an average thickness of 12.4 inches. These sludges are approximately 45% water, 40% oil, and 15% solids, by weight. This depth contrasts with sludge thickness measurements of one foot to just over three feet in September 1984 (see Appendix A of RA). An average pond sludge depth of eighteen inches is used for risk and design calculations.

Table 1-1

Descriptions of Materials Listed on Manifests for Disposal at the SDS Site

Alcohol, organic phosphorus compounds, cobalt

Alkyl Benzenes

Barge, RR Tank Car Washings, & Misc. Chemicals

Benzene, Ethers, Methyl chloride

Butyl Acrylates

Butyl Acrylates

Calcium Arsenate

Caustic and Latex Polymer

Copper Chloride Powder Catalyst

Diethylene Glycol, Resin, w/Toluene

Drilling Muds

Drum Washing Residue

Fatty Acid Esters

Fatty Alcohols

Filter Cake Residue

Furfural, Butadiene Copolymer

Still Bottoms

Herbicides

Hydraulic Oils

Insecticides

B, S & W Oils

Kerosene & Grease

Kitchen Grease & Water

Methacrylate

Molasses & Water

Oily Wastewater

Organic Sludge, Skimming, Kerosene and Mineral Spirits

Phenol Formaldehyde

Pickling Acid

Polyethylene, Diatomaceous Earth

Process Wastewater

Soap

Sodium

Sodium Hydroxide

Sour Crude Oil

Spent Chlorinated Solvents

Spent Newspaper Inks and Glycol Solvents

Styrene & Ethylbenzene Bottom

Styrene Monomer w/Diesel

Vegetable Oils

Waste Chemicals

Water & Oil

014624

L880

TABLE 1-2

SUMMARY OF REPRESENTATIVE CONCENTRATIONS AND VOLUMES FOR
SPECIFIC WASTE COMPARTMENTS AND RECEPTORS FOR THE SHERIDAN DISPOSAL SERVICES SITE

PARAMETER	REPRESENTATIVE WATER CONCENTRATIONS (1)			REPRESENTATIVE CONCENTRATIONS FOR SOILS/SLUDGES (1)					
					DIKE AREA		EVAPORATION AREA		
	MAIN POND WATER (2) (TABLE 4-1&2)(7)	CLARK LAKE WATER (WEST END) (TABLE 8-7)(7)	BRAZOS RIVER WATER QUALITY (TABLE 8-4)(7)	MAIN POND SLUDGES (2)	AVERAGED SOILS (3)	AFFECTED SOIL	SOILS	SLUDGES (2)	BACKGROUND SOILS (TABLES 4-1&2)(7)
UNITS	mg/l	mg/l	mg/l	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg
Area (acres)	19 (4)		Surficial	12	17	12	40.75	0.25	Surficial
Representative thickness (feet)	7.1 (5)		NA	1.5	11	5.5	0.5	0.5	NA
Approximate volume (cu.yds.)	2*7000		NA	30000	302000	114000	33000	200	NA
Density	1 0 g/cm3	1.0 g/cm3	1.0 g/cm3	1 1 g/cm3	1 6 g/cm3	1 3 g/cm3	1.6 g/cm3	1 3 g/cm3	1.6 g/cm3
Benzene	ND	ND	ND	1050	12	23	ND	42.95	ND
2,4-Dimethylphenol	ND	ND	ND	342	8	16	ND	24.7	ND
Ethylbenzene	ND	ND	ND	3267	6	11	ND	301	ND
Naphthalene	ND	ND	ND	257	6	12	ND	ND	0 135
Total PCBs	ND	ND	ND	159	10	19	ND (6	25	ND
Phenol	1.8	ND	ND	695	45	90	ND	757	ND
Tetrachloroethylene	0.26	ND	ND	121	ND	ND	ND	21.8	ND
Toluene	0.63	ND	ND	1225	10	19	ND	165	ND
Trichloroethylene	1.46	ND	ND	50	ND	ND	ND	ND	ND
Metals (as total constituent)									
Chromium	1.14	0 01	0.01	236	13	20	33.4	72.6	31.1
Lead	0.304	0.012	0.0235	404	149	290	45.7	164	11.6
Nickel	0.625	ND	0.005	67	12	14	17.6	31.5	17 2
Zinc	4.505	0.07	0.035	573	285	492	72.6	751	39.6

(1) Except for sludge samples, the representative concentration given is the average of [detected concentration(s) plus 1/2*(sample d.l.)] found for the waste compartment indicated.
(2) Representative values for sludge samples are calculated as the average of all above-detection-limit concentrations or 1/2 of detected value if there is only one detection in the data base. See Appendix E-1 for documentation. These calculations provide worst-case levels for direct contact exposure and ground water models. Representative values for Toluene and Zinc to not include outlier values of 36,600 and 13,800 mg/kg, respectively, because concentrations of that magnitude were not confirmed in 1987 samples.
(3) Average dike soil concentration is calculated as the mean of all visually affected and unaffected soils data from the dike boring samples collected by July 6-10, 1987 (see Appendix E-1 for documentation). These calculations provide data for dust emissions and direct exposure models.
(4) Acreage given is based on the assumption that, if abandoned, the pond would fill to an elevation of 176.5 feet MSL and would decant over the lowest part of the dike at that elevation with each subsequent rainfall. The low point appears to be located on the north dike, west of the former tank battery and receiving pond (Figure 2-2).
(5) Depth given is based on the average depth (3.6 feet) from the water surface to the sludge surface (see Table 3-5) as recorded in the June 17-18, 1987 sampling of pond sludges, plus the difference between the water elevation recorded on that date (173' MSL) and 176.5' MSL (a difference of 3.5').
(6) PCBs (as Aroclor 1248) were detected from 0.5-1' depth in evaporation system cells 3 and 15 at concentrations of 1600 ug/kg and 110 ug/kg, respectively (samples from February 1988 and December 1987 sampling events).
(7) These Table numbers refer to the Baseline Risk Assessment, November, 1988.

014625

Pond Dike

The dike around the pond has a surface area of approximately seventeen acres and was constructed primarily from surrounding clays and combustion residues from the incinerator. It is estimated that up to ten percent of the levee material may consist of materials characterized as diatomaceous earth filter aid wetted with a petroleum oil. This filter cake material contains unspecified organo-metallic chemicals as well as insoluble barium and zinc salts.

A series of borings were made through the depth of the dike to confirm its construction and to characterize soils and waste materials within it. Appendix A of the Source Control FS contains the boring logs and summary tables for the organic and inorganic analytical results. The boring logs indicate that affected soil and sludge in the dikes is typically not encountered until depths in excess of three feet.

SDS began initial closure actions in 1979. Approximately five acres of the pond in the northern section were covered with construction debris and dike material. In an earlier pond closure effort (not completed), another two acres of the southeastern portion of the pond were capped using apparently clean fill materials and on-site soils. (These seven acres are included in the estimated dike area of seventeen acres.)

Evaporation Area Sludges and Soils

The evaporation system consists of 42 acres of water retention cells. The majority of organic compounds that were identified in the evaporation system occur in two isolated sludge deposits at or near the point of pond discharge into the evaporation system. Based on samples collected in June 1987 and December 1987, the remainder of the evaporation system contains soils that are generally characteristic of background soils in the area. Metal concentrations were generally in the same range as background and no volatile or semi-volatile compounds were identified. PCBs were detected at only two of nineteen sample locations at concentrations of 1600 and 110 ug/kg (ppb). (Appendix A of the Source Control FS).

Process Tankage

Treatment process units at the SDS site are located on the top of the levee and include an incinerator, a boiler, and nine tanks. The tanks were used for separation and treatment of oil/water emulsions and storage of solvents and fuel oils. The tanks vary in size from 500 to 1000 barrels in capacity. The tanks presently contain approximately 1500 bbl of oil and emulsion removed from the surface of the pond.

014627

2 - EVALUATION OF GROUND WATER FLOW REGIME AND QUALITY

2.1 Ground Water Flow Regime

The hydraulic characteristics of the unconfined aquifer were determined by three different methods: laboratory permeability tests, slug or bail tests, and a pumping test. Results of laboratory permeability testing are discussed in more detail in Section 3.5 of the GWRI. The geometric mean coefficient of permeability of the samples from Stratum B is 1.4E-04 cm/sec.

Slug and/or bail tests were completed in all newly installed PVC wells. Field procedures and data analysis are presented in Appendix C of the GWRI. A summary of results from the GWRI indicate a range of hydraulic conductivity from 3.3E-03 to 1.4E-02 cm/sec, with a geometric mean value of 5.8E-03 cm/sec. These values are in the range for a silty sand to a sand and are expected given the lithology of the aquifer. Transmissivity ranged from 1.5E+03 to 7.5E+03 gpd/ft, with a geometric mean value of 2.9E+03 gpd/ft.

As part of the Source Control RI, a thirteen-hour pumping test was conducted on the unconfined aquifer beginning 12/18/85. Analysis of the data yielded an average hydraulic conductivity of 7.9E-03 cm/sec and transmissivity of 4.0E+03 gpd/ft, using a saturated thickness for the aquifer of 24 feet. These data also showed that the specific capacity of the unconfined aquifer is about 0.12 gpd/ft. This would support a water well of 2 gpm, based on a saturated thickness of 24 feet.

Comparison of the three sets of data show good agreement between the different methods of aquifer analysis. Permeability measurements from the laboratory are slightly lower than the field data, which may be a function of sample size and recompaction of the sample into the permeameter. Mean pumping and slug test values for permeability and transmissivity are very close given the variable nature of the methods (a localized test versus a more regional test) and the inhomogeneities of the aquifer. These three sets of data characterize the unconfined aquifer in the vicinity of the site.

In order to assess the movement of ground water in the unconfined aquifer, water level measurements have been recorded at the site on a regular basis (monthly or more frequently) from November 1985 until February 1988. Data from these measurements are presented in Table 3-8 of the GWRI.

These data indicate that the primary flow direction in the unconfined aquifer is to the north or northwest, towards the Brazos River. The flow direction occasionally changes to that of south and west across portions of the site. These changes are related to the stage (river height) of the Brazos River and the relative amount of bank storage within the unconfined aquifer.

The river stage at the site has in some cases been measured at the site and in other cases estimated from measurements taken at the USGS station in Hempstead, Texas. Details of this analysis are in the GWRI.

As noted above, careful inspection of a complete set of ground water contour maps in the GWRI yield several trends or relationships. Of primary importance is the influence of the stage of the river on the ground water flow directions and gradients. By inspection of the available data and calculated river stage at the site, it was determined that when the river reaches a stage of approximately 140 feet MSL at the site, the ground water direction shifts from its typical northerly flow to a southerly and westerly direction. The length of time that the flow stays in the southerly or westerly direction depends on such factors as the length of time the river is at a high stage and the previous water level (saturated thickness) in the unconfined aquifer.

In order to compare the relative amount of time the gradient is in any of the three general directions, data from the years 1986 and 1987 were tabulated. The northerly flow direction (including north/northeast to north/northwest) is prominent for this aquifer, that is, it occurs about 50% or more of the time. The westerly direction is next most common, occurring about 35% of the time. The remainder of flow occurs in the southerly direction. The percentage of time calculated here and in the following discussion assumes no change in flow direction between individual sampling events; the percentage is calculated as the number of occurrences of the event divided by the total number of occurrences.

For each of these directions an average gradient has been calculated and the data are summarized in Table 2-1. The average northerly gradient for 1986 and 1987 is 0.0023. In the westerly direction the gradient is 0.0018, and in the southerly direction it is 0.0058. The actual lines along which the gradients were calculated and their locations are on the ground water contour maps in the GWRI.

C253

TABLE 2-1

Average Gradient, Flow Direction and Percent of Time
for the Unconfined Aquifer (Stratum B)

Month or Month	Direction	Number of Occurrences [a]	% of Time [b]	Average Gradient
1986	N (NNW-NNE)	6	50	0.0018
	W (NW-W)	4.5	38	0.0017
	S (SW-S)	1.5	12	0.0042
1987	N	14	57	0.0028
	W	8	33	0.0018
	S	2.5	10	0.0074
Jan - July 1987	N	5	35	0.0020
	W	7	48	0.0018
	S	2.5	17	0.0074
Aug - Dec 1987	N	9	90	0.0038
	W	1	10	0.0014
	S	0	0	--
1988	N	4	100	0.0043

NOTES:

[a] Number of occurrences based on dates when flow is in indicated direction - dates when flow is bidirectional are considered half of an occurrence for each of the directions.

[b] The percentage of time calculated here assumes no change in flow direction and gradient between individual sampling events; the percentage is calculated as the number of occurrences of the event divided by the total number of occurrences.

014630

While the 140' river level at the site usually signifies a change in ground water flow direction, this relationship also depends on the level of water already existing in the aquifer. That is, when the river level reaches 140 feet it does not necessarily always change flow directions. For example, on July 15, 1987, the river was at 140.3 feet but the flow was still to the north, probably due to the water previously stored in the aquifer due to earlier abundant rains and bank storage (Table 2-1). Conversely, if the Brazos River has been low for an extended period of time (say six or more months), the direction may change before the river stage reaches 140 feet because the water level along the river is so low. The contour maps for the last months of 1987 and for 1988 show how the aquifer has been draining into the river and the elevations in the wells along the river have been decreasing with time.

The gradients in the unconfined aquifer are also tied into the relationship between bank storage and river stage. This is most readily evident along the bank of the river. When the river has been at low stage for a long time without recharge from bank storage, the gradient near the river is steeper than the average gradient by about an order of magnitude. When the river stage comes up, if it quickly goes above 140' the southerly gradient is rather steep. If, however, the river only rises to around 140' or stays there for any length of time, the gradient along the river flattens out. Again, this also depends on the head in the aquifer, such that if the head is relatively high and the river level comes up quickly, the gradient along the river would not tend to be as steep as if the head in the aquifer was lower at the beginning of the river rise.

As discussed in the GWRI, the net flow (calculated using data from 1986 and 1987) is to the north. For gradients calculated over the entire site, the net ground water flow was approximately 28 feet to the north for 1986, and 49 feet to the north for 1987. Gradients calculated between the northern edge of the main pond and the Brazos River (the area where the greatest number of changes in ground water flow direction and gradient occurs) varied considerably. In 1986 the net ground water flow was less than one foot to the south between the main pond and the river, while in 1987 the net ground water flow was 121 feet to the north in this same area. This indicates that while there is a southerly component of flow for short periods of time or over small distances, the overall flow in the unconfined aquifer is to the north, into the Brazos River.

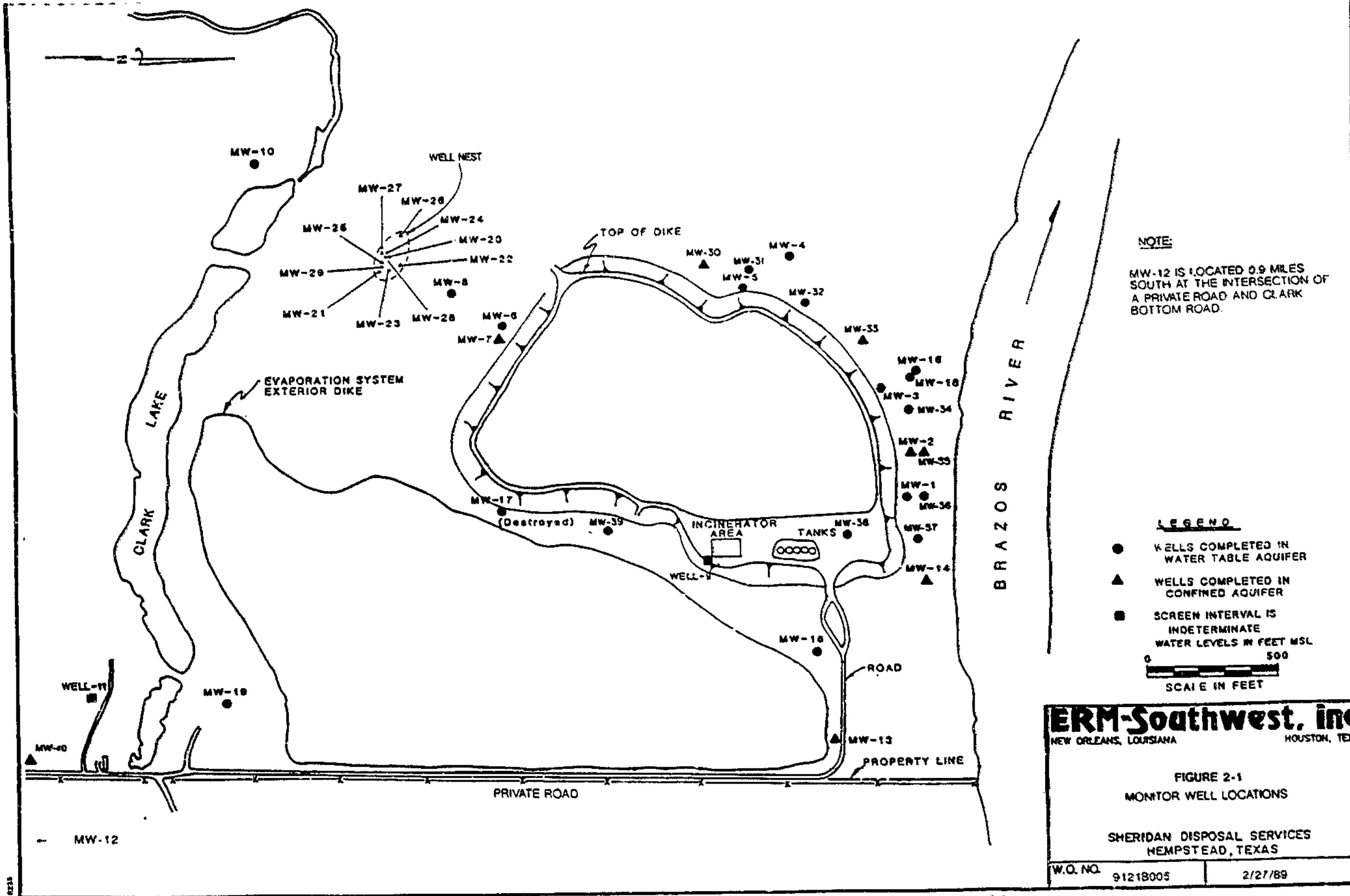
2.2 Ground Water Quality

Monitoring wells were installed at the site beginning in 1963 and ending in 1987. These wells are labelled MW-1 through MW-40 and are shown in Figure 2-1. From 1963 through 1984 different wells have been sampled, primarily for general water quality parameters such as pH, specific conductance, total dissolved solids, chloride, plus some volatile and semi-volatile analyses collected in 1984 and 1985. The quality of these data, both in terms of analytical reproducibility and methods of analysis, has been questioned. It was determined that QA/QC procedures were not adequate for these analyses. Therefore the results of these data were only used qualitatively.

As part of the GWRI, selected wells around the SDS site were sampled in October 1987 for Priority Pollutant constituents, and indicator parameters TOC, TOX, phenolics, and cyanide. Field parameters included pH and specific conductivity. Thirteen of the wells sampled were screened in the shallow unconfined aquifer. These included monitor wells MW-31, MW-32, MW-34, MW-36, MW-37, MW-38, MW-39, and MW-40, installed in 1987 for the GWRI, and wells MW-6, MW-10, MW-12, MW-18, MW-19, and MW-29 installed during 1984-1985 for the SCRI. Details of water sampling procedures are provided in the GWRI.

Analytical results of the October 1987 sampling for the shallow aquifer are presented in Table 2-2 (Complete analytical data are provided in the GWRI). For all wells sampled in the unconfined aquifer no compounds were detected in the base neutral/acid extractable group or the pesticide/PCB group. Trace concentrations of dissolved metals were detected in several of the wells. These trace concentrations are attributed to natural background variations as well as site activities. Constituent levels of these metals, however, are at or below their respective Maximum Contaminant Levels (MCLs) (Table 2-3). A discussion of the other parameters measured (e.g. TOC, TOX) can be found in the GWRI.

During the 1987 priority pollutant sampling, four compounds were identified in the volatile fraction group in levels above detection limits: benzene, tetrachloroethylene, trans-1,2-dichloroethylene, and trichloroethylene. These constituents were detected in only three monitoring wells, MW-34, MW-37 and MW-38. Wells MW-34 and MW-37 are located north and northwest of the main pond, and Well MW-38 is found along the northeastern edge of the main pond on the inside of the dike (see Figure 2-1 for well locations). Concentrations of these constituents ranged from just above



ERM-Southwest, inc
NEW ORLEANS, LOUISIANA HOUSTON, TX

FIGURE 2-1
MONITOR WELL LOCATIONS

SHERIDAN DISPOSAL SERVICES
HEMPSTEAD, TEXAS

W.O. NO. 91218005 | 2/27/89

014633

TABLE 2-2

Results of Ground Water Sampling
for Wells Screened in Unconfined Aquifer, October, 1987

Sheridan Disposal Services

ENAL 864874-			005	015	006	021	002	001	014	015
Well Number			MW-10	MW-12	MW-6	MW-18	MW-19	MW-29	MW-31	MW-32
Date Sampled			10/27/87	10/26/87	10/27/87	10/29/87	10/27/87	10/27/87	10/28/87	10/28/87
Constituents	Number of Compounds	Units	Upgradient	Upgradient	Downgradient	Downgradient	Downgradient	Downgradient	Downgradient	Downgradient
VOLATILE ORGANICS	27									
Benzene		ppb	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)
Tetrachloroethene		ppb	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)
trans-1,2-Dichloroethene		ppb	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)
Trichloroethene		ppb	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)
BASE/NEUTRAL/ACID EXTRACTABLE ORGANICS	55	ppb	ND	ND	ND	ND	ND	ND	ND	ND
PESTICIDES/PCBs	26	ppb	ND	ND	ND	ND	ND	ND	ND	ND
METALS (FILTERED)	13									
Arsenic		ppm	ND(0.003)	ND(0.003)	0.007(0.003)	ND(0.05)	ND(0.003)	0.005(0.003)	0.006(0.003)	ND(0.003)
Copper		ppm	ND(0.006)	0.012(0.006)	ND(0.006)	ND(0.006)	ND(0.006)	ND(0.006)	ND(0.006)	ND(0.006)
Selenium		ppm	ND(0.004)	ND(0.004)	ND(0.002)	ND(0.004)	ND(0.004)	ND(0.004)	ND(0.004)	ND(0.002)
Zinc		ppm	0.04(0.01)	0.06(0.01)	0.05(0.01)	0.07(0.01)	0.06(0.01)	0.02(0.01)	0.07(0.01)	0.05(0.01)
OTHER INORGANIC PARAMETERS										
Total Organic Carbon		ppm	4.7(0.1)	3.2(0.1)	3.8(0.1)	25 (0.1)	2.7(0.1)	2.5(0.1)	15(0.1)	16(0.1)
Total Organic Halogens		ugCL/L	17(5)	27(5)	17(5)	79(5)	36(5)	23(5)	120(5)	230(5)
Total Cyanide		ppm	ND(0.01)	ND(0.01)	ND(0.01)	ND(0.01)	ND(.01)	ND(.01)	ND(0.01)	ND(0.01)
Phenolics		ppm	ND(0.01)	ND(0.01)	ND(0.01)	0.01(0.01)	ND(.01)	ND(.01)	ND(0.01)	ND(0.01)
FIELD MEASUREMENTS										
pH		S.U.	6.51/6.50	6.94/6.98	6.82/6.70	6.80/6.82	6.51/6.55	6.88/6.89	6.72/6.66	6.46/6.43
Specific Conductivity		umhos/cm	883/876	1018/1022	1061/1050	1918/1922	1058/1044	839/840	1376/1326	2760/2730
Temperature		C	22	21	22.5	21.5	22	22.5	23	22.5

ENAL 864874-			020	017	018	024	012	008	022	003
Well Number			MW-34	MW-36	MW-37	MW-38	MW-39	MW-39	Field Blank	Trip Blank
Date Sampled			10/29/87	10/29/87	10/29/87	10/30/87	10/28/87	10/28/87	10/30/87	10/27/87
Constituents	Number of Compounds	Units	Downgradient	Downgradient	Downgradient	Downgradient	Downgradient	Duplicate 1	Duplicate 3	
VOLATILE ORGANICS	27									
Benzene		ppb	27(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)
Tetrachloroethene		ppb	ND(5)	ND(5)	13(5)	21(5)	ND(5)	ND(5)	ND(5)	ND(5)
trans-1,2-Dichloroethene		ppb	25(5)	ND(5)	5.2(5)	ND(5)	ND(5)	ND(5)	ND(5)	ND(5)
Trichloroethene		ppb	15(5)	ND(5)	ND(5)	13(5)	ND(5)	ND(5)	ND(5)	ND(5)
BASE/NEUTRAL/ACID EXTRACTABLE ORGANICS	55	ppb	ND	ND	ND	ND	ND	ND	ND	ND
PESTICIDES/PCBs	26	ppb	ND	ND	ND	ND	ND	ND	ND	ND
METALS (FILTERED)	13									
Arsenic		ppm	ND(0.003)	ND(0.003)	ND(0.003)	ND(0.003)	0.043(0.003)	0.048(0.003)	ND(0.003)	ND(0.003)
Copper		ppm	ND(0.006)	ND(0.006)	ND(0.006)	0.066(0.006)	ND(0.006)	ND(0.006)	0.026(0.006)	ND(0.006)
Selenium		ppm	ND(0.002)	0.010(0.004)	ND(0.004)	ND(0.004)	ND(0.004)	ND(0.004)	ND(0.002)	ND(0.002)
Zinc		ppm	0.06(0.01)	0.07(0.01)	0.04(0.01)	0.11(0.01)	0.04(0.01)	0.02(0.01)	0.03(0.01)	ND(0.01)
OTHER INORGANIC PARAMETERS										
Total Organic Carbon		ppm	42(0.1)	7.2(0.1)	16(0.1)	28(0.1)	39(0.1)	35(0.1)	0.7(0.1)	0.5(0.1)
Total Organic Halogens		ugCL/L	160(5)	20(5)	1100(5)	320(5)	310(5)	310(5)	7(5)	13(5)
Total Cyanide		ppm	ND(0.01)	ND(0.01)	ND(0.01)	ND(0.01)	ND(0.01)	ND(0.01)	ND(0.01)	ND(0.01)
Phenolics		ppm	0.04(0.01)	ND(0.01)	0.02(0.01)	ND(0.01)	0.01(0.01)	ND(0.01)	ND(0.01)	ND(0.01)
FIELD MEASUREMENTS										
pH		S.U.	6.79/6.78	6.51/6.40	6.70/6.62	6.91/6.83	6.77/6.64	---	---	---
Specific Conductivity		umhos/cm	1958/1996	1612/1605	1262/1235	1540/1530	1748/1755	---	---	---
Temperature		C	22	23	23	23	22	---	---	---

Notes:
ND = Not Detected
Detection Limits in Parentheses

014634

TABLE 2-3

List of MCLs From 40 CFR 141

Parameter	Maximum Contaminant Level
Arsenic	0.05 mg/l
Barium	1 mg/l
Cadmium	0.010 mg/l
Chromium	0.05 mg/l
Lead	0.05 mg/l
Mercury	0.002 mg/l
Nitrate (as N)	10 mg/l
Selenium	0.01 mg/l
Silver	0.05 mg/l
Fluoride	4 mg/l
Endrin	0.0002 mg/l
Lindane	0.004 mg/l
Methoxychlor	0.1 mg/l
Toxaphene	0.005 mg/l
2,4-D	0.1 mg/l
2,4,5 - TP Silvex	0.01 mg/l
Trihalomethanes (total)	0.10 mg/l
Turbidity	1-5 TU
Coliform (total)	< 1 col/100 ml
Radium 226 + 228	5 pCi/L
Gross Alpha	15 pCi/L
Beta Particle and photon radioactivity	4 mrem (annual dose equivalent)
Benzene	0.005 mg/l
Carbon Tetrachloride	0.005 mg/l
1,2-Dichloroethane	0.005 mg/l
1,1-Dichloroethylene	0.007 mg/l
para-Dichlorobenzene	0.075 mg/l
1,1,1-Trichloroethane	0.2 mg/l
Trichloroethylene	0.005 mg/l
Vinyl Chloride	0.002 mg/l

014635

G814

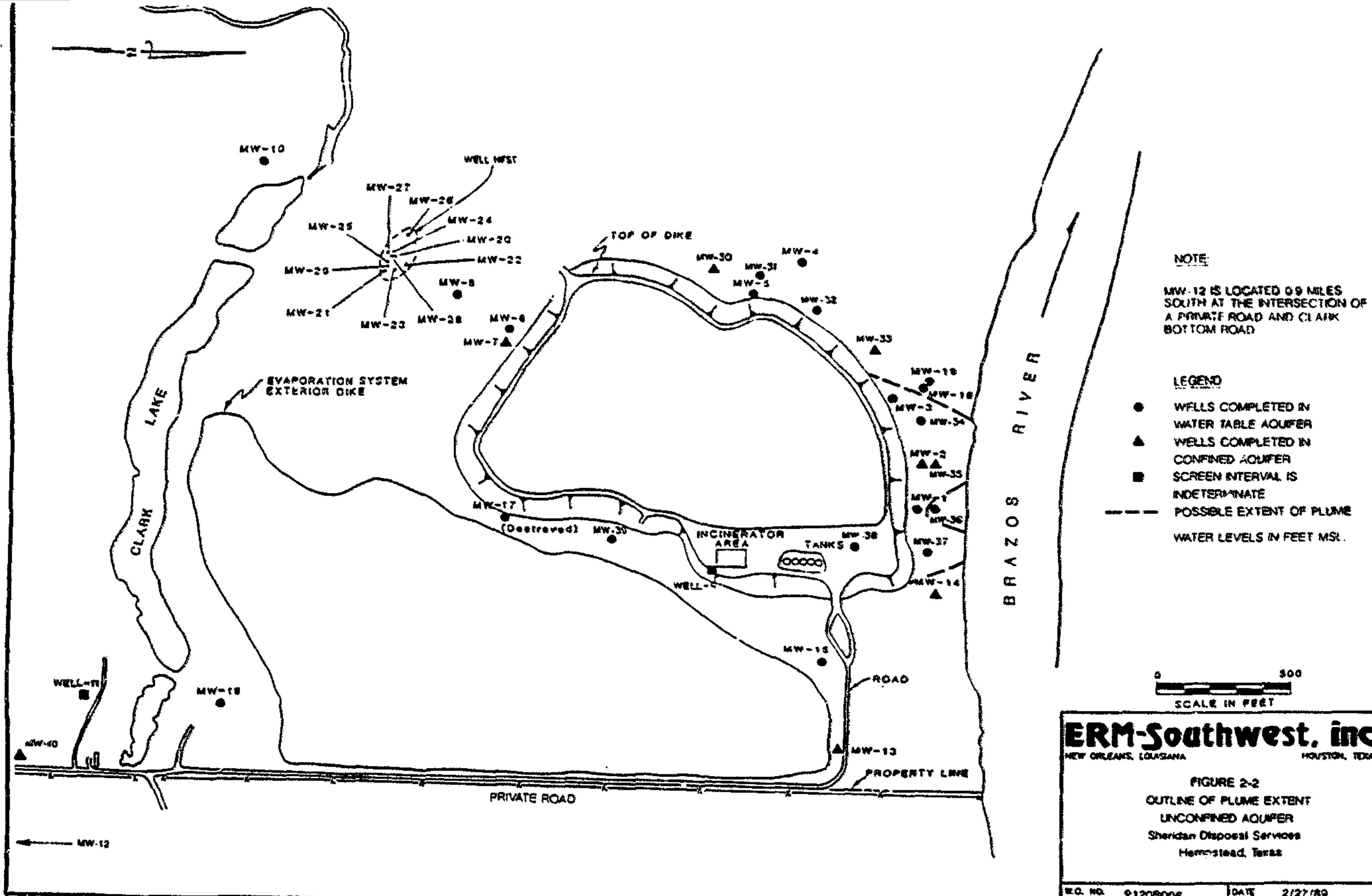
detection limits (0.0052 ppm) to a maximum of 0.027 ppm of benzene in Well MW-34.

The location and concentration of the above constituents defines a plume of low concentrations of volatile organic compounds that appear to be coming from the main pond at the SDS site and flowing north to northwest toward the Brazos River, following the general ground water flow pattern (see Figure 2-2). The vertical extent of the plume is not fully known, but it is assumed that the concentration of the above four compounds is uniform throughout the shallow unconfined aquifer. Based on all sampling results, no site constituents have reached the deeper confined aquifer beneath the site. This fact was most recently confirmed by the sampling in October 1987 which showed that none of the four volatile organic compounds (nor any of the other Priority Pollutant constituents) were detected in any of the seven wells sampled which were screened in the deeper aquifer. A complete discussion of the confined aquifer and its relationship to the shallow unconfined aquifer is found in the GWRI.

The plume volume has been approximated by taking the conservative assumptions that the area of the plume is the main source area plus the plume as shown on Figure 2-2, and that the entire saturated thickness is affected. This area was measured with a planimeter. The estimated volume was calculated as shown below.

Estimated Volume	=	Area x Aquifer Thickness x Effective Porosity
	=	1,674,444 Sq. Ft. x 24 Ft. x 0.3
	=	12,056,000 Cubic Feet
	=	90,180,000 Gallons

In summary, the analytical data from the SDS site indicate that a limited plume of volatile organic constituents exists and moves from the main pond in a northerly to northwesterly direction in response to ground water flow. Consequently, ground water with low concentrations of these constituents discharges into the Brazos River.



ERM-Southwest, inc.

NEW ORLEANS, LOUISIANA HOUSTON, TEXAS

FIGURE 2-2
OUTLINE OF PLUME EXTENT
UNCONFINED AQUIFER
Sheridan Disposal Services
Hempstead, Texas

W.O. NO.	91208006	DATE	2/27/89

014637

3 - GROUND WATER CONTROL OBJECTIVES

The NCP states the general goals and objectives of remedial actions where it defines the appropriate extent of remedies in 40 CFR 300.68(i) as: "a cost-effective remedial alternative that effectively mitigates and minimizes threats to and provides adequate protection of public health and welfare and the environment." Compliance with this overall remedial objective is measured, at least in part, by evaluating the selected alternatives' ability to mitigate site-specific risks, meet the statutory preferences for the selection of a remedy, and achieve compliance with ARARs. Criteria based on these more specific objectives are outlined below.

3.1 Risk-Based Objectives

The exposure pathways evaluated in the Risk Assessment establish the primary basis for identifying site-specific goals for each remedial alternative where existing environmental regulatory criteria are not available. Under current and most probable future land-use conditions, only one potential pathway for exposure exists for ground water; this pathway is seepage of contaminants into the shallow aquifer and subsequent discharge into the Brazos River. The Risk Assessment also describes the potential scenario of ingestion of contaminated ground water in the unlikely event that wells were to be installed between the lagoon and the river, in the future. However, it should be noted that the Source Control ROD calls for the implementation of institutional controls to prevent any well installation in the vicinity of the lagoon.

All remedial alternatives considered are designed to satisfy the following objectives:

1. Minimize potential impacts on surface waters.
2. Minimize potential for river bank erosion.
3. Minimize the potential for completion of new exposure pathways.

3.2 Section 121(b) Statutory Objectives

Section 121(b) of CERCLA, as amended by SARA, states as follows: "Remedial actions in which treatment which permanently and significantly reduces the volume, toxicity or mobility of the hazardous substances, pollutants, and contaminants is a principal element, are to be preferred over remedial actions not involving such treatment." Section 121(b) also expands the goals of remedial actions to include a preference for remedial actions that utilize permanent solutions and alternative treatment technologies or resource technologies to the maximum extent practicable.

3.3 Section 121(d) Statutory Objective (ARARs)

Section 121(d) of CERCLA, as amended by SARA, describes the types of standards that a remedial action is required to meet. The fundamental standard for evaluating remedies under Section 121 remains "protection of human health and the environment". In addition, the standards, requirements, criteria, or limitations under any Federal environmental law, or any more stringent State standard, that are "legally applicable" or "relevant and appropriate" must be met. To obtain compliance with this general standard, and in recognition of the EPA's July 9, 1987 memorandum "Interim Guidance on Compliance with Applicable or Relevant and Appropriate Requirements", remedial alternatives were analyzed to determine what regulatory requirements would be applicable or relevant and appropriate. Table 3-1 presents the universe of environmental standards that were reviewed to determine which of them had a bearing on remedial action at the site. Potentially applicable, relevant and appropriate regulations for surface water and ground water are discussed as follows:

- ARARs for Discharge to Surface Water

The Brazos River runs adjacent to the site. Ground water discharges into the Brazos River.

State water quality standards are legally enforceable counterparts to the Federal water quality criteria. In Texas, the State water quality standards are set forth in Chapters 319 and 307 of the rules and regulations of the Texas Water Commission. Those standards establish certain numerical criteria which are legally applicable to waters in the Brazos. All remedial alternatives are designed to satisfy the requirements of 31 TAC Sections 319.21-29, 307.1 to 307.10 for the discharge of water from the upper unconfined sand zone to the Brazos.

With respect to concentrations of chemicals in the river:

(1) Final Maximum Contaminant Levels (MCLs) are considered relevant and appropriate where MCLs are available; and

Table 3-1

STANDARDS, REQUIREMENTS, CRITERIA, OR LIMITATIONS EVALUATED FOR ARARs DETERMINATION

- Safe Drinking Water Act
- Clean Water Act
- Occupational Safety and Health Act
- Fish and Wildlife Coordination Act
- Endangered Species Act
- Rivers and Harbors Act of 1899
- Scenic River Act
- Texas Water Code
- Texas Water Quality Standards
- Marine Protection, Research and Sanctions Act
- Executive Order Requirements for Flood Plains and Wetlands

014640

(2) State and Federal water quality criteria for the protection of human health and the environment are relevant and appropriate where MCLs are not available.

In order to set the concentration limits for surface water different criteria are utilized, depending on the data available. Numerical criteria are set only for the four compounds detected

in the ground water, (benzene, trichloroethylene, trans-1,2-dichloroethylene and tetrachloroethylene), as the source remedy will act to prevent migration of new waste constituents. Two compounds, benzene and trichloroethylene, have Federally established MCLs which govern the level allowed in public drinking water sources. For these two compounds the MCLs are 0.005 mg/l. The other two compounds do not have established MCLs. For tetrachloroethylene, the Water Quality Criterion of 0.008 mg/l based on protection of human health at the 10^{-6} excess risk level for cancer is used. No MCLs, water quality or RfDs have been established for trans-1, 2-dichloroethylene, therefore no exposure levels could be calculated for this compound. Instead, the maximum allowable exposure concentration for trans-1,2-dichloroethylene was set at 0.005 mg/l, the same level as for the potential carcinogens benzene and trichloroethylene. Since trans-1,2-dichloroethylene is not a carcinogen, using this value will be most protective of human health. The governing concentration limits for each compound of concern are presented in Table 3-2.

o <u>ARARs for Ground Water</u>

The EPA's ground water protection strategy is based on the "differential protection" of ground water (i.e., ground water protection as it relates to a specific classification of an aquifer). Under the strategy, ground waters are classified as follows:

- o Class I - ground waters that are highly vulnerable and either an irreplaceable source of drinking water or ecologically vital;
- o Class II - ground waters currently used or potentially available for drinking water or other beneficial use; and
- o Class III - ground waters not a potential source of drinking water and of limited beneficial use.

For Class I and Class II ground water Maximum Contaminant Levels (MCLs) established under the Safe Drinking Water Act would be applicable for ground water sources which qualify as a public

Table 3-2

ARAR's for Surface Water
Sheridan Disposal Services Site

		Clean Water Act				
		Ambient Water Quality Criteria Human Health [2]		Ambient Water Quality Criteria for Protection of Aquatic Organisms		
				Freshwater		
Chemical and CAS Number	SDWA [1] MCL	Consumption of Water and Fish	Consumption of Fish Only	Acute LEC	Chronic LEC	Governing ARAR Concentrations [3]
Benzene 71-43-2	0.005	0.0066	0.4000	5.30	NA	0.005
Tetrachloroethylene 127-18-4	NA	0.008	0.0885	5.28	0.840	0.008
1,2-dichloroethylene (Trans) 540-60-5	NA	NA	NA	NA	NA	0.005 [4]
Trichloroethylene 79-01-6	0.005	0.027	0.807	45.4	NA	0.005

Notes:

NA = Not Available or Not Applicable

[1] Maximum Contaminant Level, in mg/l under Safe Drinking Water Act.

[2] Water Quality Criteria for protection of human health at the 1 E-05 incremental increase of cancer risk, mg/l.

[3] The most protective exposure level (i.e. lowest concentration) tabulated for the specific hazardous constituent based on the various regulations, mg/l.

[4] Level selected based on similar compounds which are carcinogens, although trans 1,2 dichloroethylene is not a known carcinogen.

014642

water system or a community water system. MCLs may also be relevant and appropriate to ground water that would not currently qualify as such systems but could potentially so qualify in the future. Similarly, where the State has established drinking water standards that are more stringent than the Federal MCLs, these may be applicable or relevant and appropriate.

For purposes of determining degree of remediation for ground water under Section 121(d), "a process for establishing alternate concentration limits to those otherwise applicable for hazardous constituents in ground water" exists. These alternate concentration limits which may be higher than other standards or statutory limitation (such as MCLs), are based on protection of human health. The human health based concentration limits may be utilized instead where the following criteria are met (Section 121(d)(2)(B)(ii)): 1) There are known and projected points of entry of the ground water into the surface water, 2) there is and will be no statistically significant increase in the levels of constituents from the ground water into the surface water, and 3) remediation of the site will include institutional measures to preclude human exposure to affected ground water between the facility and all known or projected points of entry of the ground water into the surface water. All of the above criteria are met for the SDS site. Alternate concentration limits (ACLs) for ground water which are protective of human health and the environment are calculated by setting the Brazos River concentrations equal to the final MCL or where not available, water quality criteria for the protection of human health. The calculation of ACLs for the compounds of concern is presented in Table 3-3, based on Brazos River low flow of 137 cfs and taking the discharge area of the plume as the entire width of the main pond parallel to the river by the total thickness of the aquifer.

There are two water-bearing zones underlying the site. The uppermost zone is unconfined. The next zone, which is separated from the upper zone by a clay aquitard, is referred to as the confined aquifer. Where the potential ground water pathway of concern is through a surface water discharge, risk-based numbers often form the basis for establishing protective levels for the saturated zone. This approach is also utilized where MCLs are not appropriate. Specific factors that may influence the appropriate risk level include:

(1) Feasibility of providing an alternative water supply;

(2) Current use of the ground water;

L563

TABLE 3-3

Calculation of Aquifer Concentrations
Based on Meeting ARAR's
in the Brazos River

Assumptions

Assumed facility width = 284.7 m (934 ft.)
Assumed aquifer thickness = 7.3 m (24 ft.)

1. Aquifer Flow (Q) = kiA
   where: k = 7.9E-03 cm/sec = 6.83 m/day
   i = 0.0023 m/m (average gradient for 1986 and 1987)
   A = 2083.4 m2 (facility width * aquifer thickness)
   Q = 32 m3/day

2. Low River Flow (at Hempstead) = 137 cfs = 335,181 m3/day.
3. Use a safety factor of 2 to calculate allowable concentrations in the aquifer.

Calculations for a Concentration of 0.005 mg/l

(Benzene, trans-1,2-Dichloroethylene, Trichloroethylene)

0.005 mg/l (river concentration) * 335,181 m3/day (min. river flow)
* 1,000 l/m3 (conversion factor) = 1,675,905 mg/day (total mass load from aquifer)

1,675,905 mg/day (mass load) / 32 m3/day (aquifer flow) * m3/1,000 l (conversion factor)

= 52.37 mg/l÷2 (safety factor)
= 26.2 mg/l = the allowable concentration in the aquifer

Calculations for a Concentration of 0.008 mg/l (Tetrachloroethylene)

0.008 mg/l (river concentration for tetrachloroethylene) * 335,181 m3/day (minimum river flow)

* 1,000 l/m3 (conversion factor) = 2,681,448 mg/day (total mass load from aquifer)

2,681,448 mg/day (mass load) / 32 m3/day (aquifer flow) * m3/1,000 l (conversion factor)

= 83.8 mg/l÷2 (safety factor)

= 41.9 mg/l = the allowable concentration in the aquifer

Notes:

Calculations for aquifer permeability, gradient and area can be found in the Baseline Risk Assessment dated November 1, 1988.

Discharge of the Brazos River at low flow was measured by the USGS at the Hempstead station, approximately 14 miles downstream of the site.

(3) Effectiveness and reliability of institutional controls;

(4) Ability to monitor and control the movement of contaminants in the ground water.

Also factored into decision making should be:

(1) Ability to limit extent of contamination;

(2) Impact of contamination on environmental receptors;

(3) Technical practicability and cost of remedial alternatives.

Clearly, MCLs are not legally applicable to the shallow unconfined ground water source at the Sheridan site. This is not a "public water system" as defined under 40 CFR Part 141.2(e), as it is not a drinking water source being supplied to at least 25 individuals at least 60 days out of the year. Indeed, this source is not supplied to any individuals, any days of the year, and institutional controls will be implemented to prevent its use in the future.

The inapplicability of MCLs does not mean that this ground water source does not need to be protected to levels that will avoid an endangerment to human health and the environment. Since the only receptor for this ground water source is the Brazos River, it is expected that this goal can be achieved by ensuring that any potential impact from the site on the ground water will not result in levels of constituents that, once discharged to the river, would have an adverse impact on human or aquatic receptors. Therefore, the calculated ACLs for ground water are based on meeting ARARs in the Brazos River. The numerical criteria for both surface and ground water based on the preceding ARARs analysis are summarized in Table 3-4. It should be noted that while these proposed ACLs will provide for meeting MCLs in the river, MCLs for benzene and trichloroethylene are currently exceeded in shallow ground water at the site.

Table 3-4

Numerical Criteria for Surface and Ground Water

Compound	Surface Water Concentration (mg/l)	Ground Water ACL (mg/l)	Ground Water MCLs mg/l
Benzene	0.005	26	0.005
Tetrachloroethylene	0.008	41	NA
1,2-trans-dichloroethylene	0.005	26	NA
Trichloroethylene	0.005	26	0.005

4 - SCREENING OF GROUND WATER REMEDIAL ALTERNATIVE TECHNOLOGIES

4.1 Purpose and Scope

The purpose of this section is to define the general classes of response actions applicable to the site, identify specific technologies under each of the general classes, and to screen applicable technologies against site-specific criteria. The technologies which are retained from this screening will be assembled into remedial alternatives and analyzed in further detail.

General response actions are classes of remedial actions which define the basic approach to solving a particular problem or group of problems at the site, but do not identify the specific technologies. General response actions are identified as applicable if they have the potential to contribute to ground water remediation either alone or in conjunction with other response actions. The general classes of remedial actions considered applicable to the ground water are:

1) Active restoration,
2) Containment through hydraulic control, and
3) Limited or not active response.

Specific methods to accomplish each of these classes of response actions have been identified. Potentially applicable methods of accomplishing the general response actions for ground water are presented in Table 4-1. This list constitutes the extent of methods considered in the screening performed in the following section.

4.2 Suitable Remedial Responses

In the next step, remedial technologies, corresponding process options, and applicable general response actions were identified. These remedial technologies and other response actions were screened in a process involving five considerations: the state of technology development, performance record, inherent construction and operation problems, site conditions, and waste characteristics. Innovative technologies that were potentially cost effective were preserved. Technologies and process options were assessed independently without regard to potential advantages and disadvantages of technologies when applied in combination. Technologies and other response actions were assessed based on their direct suitability to existing conditions at the site. The

014647

TABLE 4-1

GENERAL RESPONSE ACTIONS

Plume Containment

- o Slurry Wall
- o Grout Curtain
- o Sheet Piling
- o Pumping Wells
- o Interceptor Trench

Active Restoration

- o Biorestoration
- o In-Situ Chemical Oxidation
- o Extraction and Treatment

Limited or No Active Response

- o Natural Attenuation with Monitoring
- o Provision of Alternate Water Supply
- o Institutional Controls

results of this screening of technologies and other response actions for the Sheridan Disposal Services site are summarized in Table 4-2. Remedial technologies are grouped by the general response action which they address. Each potential remedial response is briefly evaluated in the following paragraphs.

4.2.1 Containment

Slurry Wall

A slurry wall is a vertical low-permeability barrier to ground water flow constructed in soil by excavating a trench and filling it with a soil-cement or soil-bentonite slurry. The slurry plugs the void spaces in the surrounding soils, forming the barrier to flow.

Slurry walls are a proven technology for the control of ground water in both environmental and geotechnical projects. Although a specially skilled and equipped construction crew is required, a slurry wall can be fairly easily constructed using common earth moving equipment. For the Sheridan site, depths of 50-60 feet would be required (keyed-in to Stratum C). Since construction of slurry walls to depths in excess of 100 feet has been done, it is quite certain that a slurry wall of this depth can be constructed. Once constructed, the operating and maintenance costs of a slurry wall are relatively small. Waste characteristics do not appear to be incompatible with slurry wall construction requirements.

However, slurry walls are not generally suitable for steeply sloping terrains. A slurry wall at the Sheridan site would be located near a sharp embankment above the Brazos River. Although there is a relatively flat area about 100 to 200 feet wide available for construction of a slurry wall, the potential for erosion exists.

Since erosion control will be implemented as part of the source remedy, long-term erosion will not damage the effectiveness of a slurry wall. A spur jetty erosion control system has been engineered for the dimensions and velocities of this reach of the Brazos River.

The spur jetty system reduces the velocity of water at the base of the high bank and redirects currents into the middle of the river. This prevents additional erosion and causes deposition of a protective mass of waterborne material at the base of the high bank. These spur jetty systems have been successfully used at over fifteen sites on the Brazos River since 1961.

TABLE 4-2

SCREENING OF RESPONSE ACTIONS

Response Action	Suitable
1. Plume Containment	
o Slurry Wall	yes
o Grout Curtain	no
o Sheet Piling	no
o Pumping Wells	yes
o Interceptor Trench	yes
2. Active Restoration	
o Biorestoration	no
o In-Situ Chemical Oxidation	no
o Extraction and Treatment	yes
Air Stripping	no
Carbon Adsorption	yes
Biological Treatment	no
Ozonation/Chemical Oxidation	no
3. Limited or No Active Response	
o Natural Attenuation	yes
o Provision of Alternate Water Supply	yes
o Institutional Controls	yes

For the reasons listed above, the slurry wall is a suitable technology for the Sheridan site at the screening level.

Grout Curtain

A grout curtain is a form of vertical flow barrier. It is constructed by drilling a series of boreholes in a staggered, double row pattern, and injecting grout into the formation through these boreholes. The staggered pattern results in a series of overlapping grouted columns.

The ability of grout to penetrate fine grained soils is not well demonstrated. The failure of grout penetration will leave unsealed pathways through the hydraulic barrier. The grout curtain is very expensive to construct when compared to the slurry wall and is frequently a less effective barrier to migration.

Due to the uncertainty in performance and potential construction problems stated above, the grout curtain is not retained for further consideration.

Sheet Piling

A sheet piling cutoff wall uses interlocking sheet piles, usually steel, which are driven into a lower confining layer to form a barrier to ground water flow.

The sheet piling cutoff wall, although easily implemented in the unconsolidated soils at the site does not provide an effective barrier. The joints between the interlocking piles provide a potential pathway for migration. Furthermore, the integrity of the wall may decrease with time as the steel may be corroded by the contact with ground water of relatively high conductivity.

Sheet piling is not a suitable for the site due to anticipated problems with its performance.

Pumping Wells

A recovery well may be utilized to control migration of a plume by pumping affected ground water from the aquifer. Usually a line of wells is required in order to develop a zone of influence which will control migration over a distance.

The installation and operation of recovery wells are well established and well understood technologies. With careful placement, a number of recovery wells should perform the required level of recovery. The recovered ground water would be treated prior to discharge.

The potential disadvantage of recovery wells is that the entire site discharges only an estimated 7.1 gallons per minute of ground water to the Brazos River. Wells would not be expected to recover very much flow; spacing of wells must be carefully designed so that isolated flow pathways between wells would not exist.

Recovery wells are retained for further evaluation based on the well established nature of the technology and the anticipated level of performance.

Interceptor Trench/French Drain

An interceptor trench, like recovery wells, is designed to limit the migration of affected ground water. It is a vertical excavation usually down to the bottom of the saturated zone, filled with a coarse graded backfill to promote drainage and often containing a perforated drain pipe. The recovery trench has the advantage of being a complete barrier to forward flow of ground water. The trench is typically more expensive than a line of wells, but is frequently preferred in lower permeability strata or where very low flows are to be recovered; when dense non-aqueous liquids are to be recovered; or when the stratum is non-homogenous and isolated flow pathways that will bypass a line of wells may exist.

The construction and operation of recovery trenches are well established, well understood technologies. Typical recovery trenches are constructed to a depth of about 30 feet, although with more sophisticated construction techniques, deeper trenches have been installed. A recovery trench at the Sheridan site would need to be about 60 feet deep, and would be constructed in unconsolidated sediments. In order to construct the trench some method of preventing collapse of the unconsolidated soils during excavation would have to be included.

The bio-polymer slurry trench is a method that has been successfully used in a few similar instances. The construction method is similar to that of a slurry trench in that the excavation is held open by the use of a slurry until the trench can be backfilled with the drainage media. The slurry used in this case is a biodegradable slurry, which is initially a pseudoplastic fluid, then breaks down naturally to innocuous substances. It may be feasible to construct a recovery trench at the Sheridan site using this technique.

The recovery trench is retained for further evaluation, due to the potential advantages it poses over recovery wells.

4.2.2 Active Restoration

Biorestoration

This process consists of the enhancement of natural biological degradation of organic constituents by the injection of nutrients, such as nitrogen, phosphorus and frequently oxygen and microorganisms.

The implementation of this process is fairly simple and well documented for hydrocarbon constituents; however, the success with regards to chlorinated solvents, especially in the low part per billion concentrations is unknown. In addition, areas where silts and clays predominate make transfer of materials difficult.

014653

This treatment is considered unsuitable for further consideration due to the low concentrations of not easily biodegraded constituents.

In-Situ Chemical Oxidation

This process is similar to in-situ biological treatment in that chemicals are injected into the ground water to effect treatment. A chemical oxidizer such as hydrogen peroxide is injected and chemically oxidizes the constituents to a non-toxic form.

The implementation of this process is fairly simple in permeable material, but relatively difficult in low permeability materials. It has not been widely applied as an in-situ process, therefore it is not a well developed technology.

This treatment is considered unsuitable for further consideration for several reasons listed below:

- This technology has not been widely utilized.
- It is not well developed.
- The low ground water transmissivity would slow the overall remediation.

- The process is non-specific for the target compounds. Other compounds present may interfere with the oxidation of target compounds.
- There are concerns for worker health and safety with handling chemical oxidants, which are frequently reactive or corrosive.
- The target compounds are present at very low concentrations compared to wastewater where chemical oxidation is typically used. Thus, there exists uncertainty that adequate contact of target organics with the oxidant will occur.

Extraction and Surface Treatment

1. Air stripping

In this process, the water to be treated is contacted in a countercurrent manner with air in a packed tower. Volatile constituents in the water evaporate and are carried out with the air stream. Air stripping is frequently employed where VOC concentrations are fairly high, where the high usage rate of granular activated carbon would be prohibitively expensive. The compounds in the air stream may be vented to the air if the mass flow rate or concentration meets ambient air quality standards; otherwise an activated carbon adsorber is employed to remove these compounds from the air stream before venting to the atmosphere.

Air stripping is generally a good treatment process for ground water. However, given the low concentrations of constituents in the shallow ground water at the Sheridan site, air stripping would not be certain to attain the treatment goals. Furthermore, economics favor the use of granular activated carbon adsorption at the Sheridan site. Air stripping is rejected in favor of GAC adsorption since it would be less reliable and more expensive at the Sheridan site.

2. Carbon adsorption

Granular activated carbon (GAC) adsorption is a treatment process which removes organic constituents. Active sites on the carbon physically adsorb organic constituents from the water. Eventually the active sites are occupied and the carbon must be replaced or regenerated.

The GAC treatment process is simple to implement and operate. The technology is well established. For the low molecular weight chlorinated compounds and benzene, GAC is the most reliable treatment process. Furthermore, the low concentrations of constituents to be removed make the economics most favorable for GAC adsorption. As the cost estimates in Section 6 indicate, four 200-pound disposable units costing $600 each will be utilized annually. For these reasons, GAC is retained for further study.

3. Biological treatment

Surface biological treatment consists of the biological degradation of organic constituents in a process unit, enhanced by the addition of nutrients, such as nitrogen, phosphorus and oxygen.

This treatment is considered unsuitable for further consideration due to the low concentrations of not easily biodegraded constituents.

4. Ozonation/Chemical Oxidation

The organics in the ground water are chemically oxidized to a less toxic form in a continuous flow, stirred tank reactor with the addition of a chemical oxidizer such as hydrogen peroxide, or UV catalyzed ozone. The anticipated reaction products are carbon dioxide, water, and hydrochloric acid. A pH adjustment system could potentially be required to neutralize the end products; also a vent gas scrubber may be required to control acid gas release from the reaction.

The most commonly used oxidants are hydrogen peroxide and ozone, frequently UV catalyzed. Reported uses of hydrogen peroxide are mainly for treatment of inorganics. Studies indicate that catalyzed hydrogen peroxide oxidation processes can remove organics by 36% to 66% (Removal of Hazardous Wastes in Wastewater Treatment Facilities - Halogenated Organics, WPCF 1986). Removal of benzene to 0.005 mg/l would require an efficiency greater than 80%. Hydrogen peroxide oxidation systems have the added disadvantages of complex operation, interferences of other constituents with the reaction, the hazard and expense of the reagent ($0.60/lb in 1988), and frequent requirement for metals addition as catalyst. For these reasons, hydrogen peroxide oxidation is not retained.

Ozonation is a process similar to hydrogen peroxide oxidation. Ozone is an unstable gas and must be generated on-site, increasing the complexity of the system. Ozone is unreactive or very slow to react with many chlorinated aliphatic compounds, including tri- and tetrachloroethylene. Ozonation is also relatively ineffective at benzene oxidation. This process has added disadvantages of high capital and O&M costs, and requirement of skilled treatment plant operators to maintain and operate the sophisticated equipment. (Ibid pp. 61-68). For these reasons, ozonation is not a suitable treatment process for the Sheridan site.

As with in-situ chemical oxidation, this treatment is considered unsuitable for further consideration due to the low concentrations of constituents and unfavorable reaction kinetics. Also, this type of system is more complex to operate and less reliable than other treatment schemes.

4.2.3 Other Remedial Responses

Natural Attenuation

Natural attenuation consists of various naturally occurring physical, chemical and biological processes which act to reduce constituent levels. These processes may include adsorption on the soil matrix, biodegradation, volatilization and dilution with incident rainwater. Natural attenuation is generally applicable under the following conditions.

- Low aquifer transmissivity (less than 50 ft^2/day)
- Low concentrations of contaminants
- Low potential for exposure
- Low projected demand for future use of the ground water

According to EPA guidance, "when constituents are expected to attenuate to health based levels in a relatively short distance or when there is a narrow strip of land between the discharge stream where contaminant levels are not expected to increase, natural attenuation may be the most practicable response." (Guidance on Remedial Actions for Contaminated Ground Water at Superfund Sites, EPA/540/6-88/003 pp. 5-7 to 5-9). Since most of the preceding conditions apply, natural attenuation is retained for further consideration.

Institutional Controls

The restriction of ground water use is generally accomplished through the use of institutional controls. Institutional controls

01465 6

proposed herein include restriction of construction of water wells and deed restrictions recorded in the county clerk's office on the use of the shallow aquifer between the pond and the river. Institutional controls will be sufficient because the land owner has agreed not to take any action which would adversely affect the remedy.

The use of institutional controls to prevent use of affected ground water is anticipated to be reliable and effective for the following reasons:

- There are no current users of shallow ground water downgradient of the site between the pond and the river (see Figure 2-3 of the GWRI).
- The narrow strip of land between the site and the Brazos River is part of the Sheridan property.
- The plume is now located in this narrow strip of land from the main pond due north to the Brazos River, with limited migration in the east-west direction.
- The measured hydraulic gradients indicate that the prevailing flow is the north (to the Brazos River).
- The amount of water per unit width (perpendicular to the flow direction) in the unconfined aquifer is very low, approximately 2.2 ft^2/day (17 gpd/ft).
- The yield of this aquifer is too low to be of agricultural use, which is the most likely potential use.

It can be concluded that ground water use from the affected area can be effectively restricted by preventing site access and by deed restrictions recorded in the county clerk's office. For this reason, institutional controls are retained for further consideration.

Monitoring

Monitoring consists of the analysis of ground water samples from wells both upgradient and downgradient of the plume to track its movement. Additionally, for the Sheridan site, monitoring of surface water is appropriate since the plume has reached the Brazos River. Monitoring is used to determine the effectiveness of active restoration processes as well as natural attenuation with institutional controls. It is easily accomplished through the use of monitoring wells. Monitoring for this site is not only feasible, but is a necessary component of every action alternative. It is retained for this reason.

5 - ASSEMBLY OF GROUND WATER REMEDIAL ALTERNATIVES

The objective of this task is to combine surviving technologies from Section 4 into a range of remedial alternatives for the SDS site which focus on the remediation objectives presented in Section 3, and which are consistent with EPA requirements and sound engineering practice. A total of five remedial action alternatives were developed, including a no-action alternative. A fact sheet for each alternative provides a discussion of the disposition of site material along with the sequence of the proposed remedial work. This information is used along with considerations of effectiveness, implementability and cost to select a more limited set of alternatives for detailed analysis.

5.1 Assembly of Alternatives

From the list of suitable remedial action technologies contained in Table 4-2, it is possible to assemble complete remedial alternatives which address the remedial objectives in Section 3, attain Federal and State requirements that are applicable or relevant and appropriate and are protective of human health and the environment.

The EPA guidance document (Guidance on Remedial Actions for Contaminated Ground Water at Superfund Sites) issued since the passage of the Superfund Amendments and Reauthorization Act sets forth a general scheme for developing a range of remedial alternatives that should be evaluated in the FS. The general categories within this range are as follows:

1. A No Action alternative.

2. A natural attenuation alternative that includes institutional controls and monitoring.

3. An active restoration alternative that reduces contaminant levels to required cleanup levels in the minimal time feasible.

4. Additional active restoration alternatives that achieve cleanup levels over longer time frames.

5. A plume containment alternative that prevents expansion of the plume.

6. An alternative involving wellhead treatment or provision of an alternative water supply and institutional controls when active restoration is not practicable.

The principal feature at the Sheridan Disposal Services site is the twelve acre surface impoundment or main pond. The source remedy includes treatment of sludges and affected soils from this impoundment, and disposing of treatment residues along with other source material under an engineered cap. Since the source material will be effectively contained, the ground water alternatives focus on the remedy of existing waste constituents in the ground water.

5.2 Remedial Alternatives

The following pages contain fact sheets for each alternative.

o <u>Alternative 1 - No Action</u>

Sequence of Work

None. No monitoring, inspection or maintenance.

Discussion:

This alternative does not provide for any additional capital improvements at the site, beyond the source area treatment and containment.

o <u>Alternative 2 - Natural Attenuation with Institutional Controls and Monitoring</u>

Sequence of Work

1. Monitor ground water and Brazos River water quality

2. Implement institutional controls

Discussion:

This alternative provides for allowing natural attenuation processes to act upon the constituents, applying institutional controls to prevent ground water from domestic or agricultural use and monitoring the quality of water within the current ground water plume. As demonstrated in Section 3 of this document, the low concentrations detected in the unconfined aquifer do not pose a current risk to human health and the environment. The implementation of institutional controls effectively addresses the future exposure to ground water. Furthermore, since the source areas will be effectively contained, future site conditions should not deteriorate. Therefore, both current and future conditions are addressed with this alternative.

o <u>Alternative 3 - Partial Slurry Wall with Ground Water Treatment</u>

Sequence of Work

1. Install slurry wall downgradient of the site (i.e. between the site and the Brazos River).

2. Install recovery wells upgradient from the slurry wall.

3. Construct surface treatment facility.

4. Recover and treat on-site and discharge.

5. Monitor ground water.

6. Implement institutional controls.

Discussion:

In this alternative, the primary mechanism for controlling migration of waste constituents in the ground water is the slurry wall, with the recovery wells collecting flow around the ends of the wall. The presence of the slurry wall minimizes the withdrawal of unaffected water with the wells, thus minimizing the volume of water requiring treatment.

o <u>Alternative 4 - Complete Slurry Wall</u>

Sequence of Work

1. Construct slurry wall around the site (i.e. around the main pond).

2. Dewater the area inside the excavation for the slurry wall. Dispose of affected ground water on-site in a manner approved by EPA or at an off-site facility permitted to accept CERCLA site wastes.

3. Install leachate collection system.

4. Recover leachate and dispose on-site in a manner approved by EPA or at an off-site facility permitted to accept CERCLA site wastes.

5. Monitor ground water.

Discussion:

The slurry wall will act to surround the source and attainment area, eliminating migration of ground water through any remaining materials. A properly designed and constructed slurry wall will provide an effective ground water barrier for many decades with little or no maintenance. The very minor amount of water removed after construction consists of incident rain water percolating through the area and a small volume of ground water flowing through the low permeability wall. Removing this water through a leachate collection system ensures that the hydraulic gradient is inward.

o <u>Alternative 5 - Recovery Wells and Ground Water Treatment</u>

Sequence of Work

1. Install recovery wells downgradient of site.
2. Construct surface treatment facility.
3. Recover and treat ground water on-site.
4. Monitor ground water.
5. Implement institutional controls.

Discussion:

This alternative includes the traditional method of control of ground water migration: withdrawal and treatment. The treated water will be discharged to the Brazos River.

o <u>Alternative 6 - Recovery Trench and Ground Water Treatment</u>

Sequence of Work

1. Install recovery trench downgradient of site.
2. Construct surface treatment facility.
3. Recover, treat and discharge ground water on-site.
4. Monitor ground water.
5. Implement institutional controls.

Discussion:

This alternative is the same as Alternative 5 except that the ground water is recovered with a trench rather than wells.

014665

5.3 Initial Screening

Following assembly of these six remedial alternatives, each alternative was evaluated on the basis of effectiveness, ease of implementation, and preliminary costs. Preliminary cost estimates are presented in Table 5-1. On the basis of this evaluation, two alternatives were rejected. The rationale for rejecting these alternatives is provided below:

Alternative 4 - Complete Slurry Wall

This alternative effectively addresses all the risk-based and regulatory remedial objectives. However, a slurry wall encircling the entire site is the most expensive alternative, while being no more effective at ground water remediation than other less expensive alternatives such as Partial Slurry Wall with Ground Water Treatment and Recovery Wells with Ground Water Treatment.

Alternative 6 - Recovery Trench with Ground Water Treatment

This alternative also effectively addresses all the risk based and regulatory remedial objectives. It is screened out at this stage based on both costs and implementability. The problem of excavating a relatively deep trench in unconsolidated, non-cohesive soils has been successfully addressed at a few sites by the use of a biodegradable polymer to hold the trench open until it is backfilled. A very few, highly specialized construction firms are currently able to perform this type of work. There is limited space at this site in which to complete this construction. This type of trench can be constructed, but its installation would be difficult, more expensive, and no more effective than utilizing wells to recover the ground water.

5.4 Summary

The following alternatives survived the preceding initial screening, and undergo detailed design analysis in the next section:

- No Action
- Natural Attenuation with Institutional Controls and Monitoring
- Partial Slurry Wall with Ground Water Treatment
- Recovery Wells with Ground Water Treatment

Table 5-1

Comparison of Cost and Time to Completion

Alternative	Total Cost [a] (Million $)	Time to Completion (Years)[b]
1. No Action	---	--
2. Natural Attenuation with Institutional Controls and Monitoring	0.3	30
3. Partial Slurry Wall w/ Ground Water Treatment	4.2	25
4. Complete Slurry Wall	10.8	25[c]
5. Recovery Wells w/ Ground Water Treatment	5.3	25
6. Recovery Trench w/ Ground Water Treatment	8.3	25[c]

[a] For monitoring costs, it is assumed that wells are installed during Source Remediation. Costs included here are only the cost of sampling and analyses.

[b] The time frames calculated in Appendix A are only general approximations of the time it might actually take to extract one pore volume. Actual time frames may be considerably longer. The costs and times to completion are based on the assumption that extraction of one pore volume will be adequate for completion.

[c] Since these alternatives were not carried through to the design analysis, specific times to completion were not calculated. The time frames for these alternatives are assumed to be the same as for Alternative 5.

6 - DETAILED ANALYSIS OF GROUND WATER ALTERNATIVES

Previous sections identified combinations of ground water control technologies that can be used at the SDS Site to protect human health and the environment. Section 5 developed these combinations of technologies into alternatives and screened out two alternatives. This section develops further the evaluation of the surviving alternatives, and then compares the relative strengths and weaknesses of each alternative. The remaining alternatives are designated as follows for ease of reference:

Alternative A -	No Action
Alternative B -	Natural Attenuation with Institutional Controls and Monitoring
Alternative C -	Partial Slurry Wall with Ground Water Treatment
Alternative D -	Recovery Wells with Ground Water Treatment

Comparisons of the detailed design of the three remaining alternatives are made in terms of compliance with ARARs; reduction in toxicity, mobility or volume; short-term effectiveness; long-term effectiveness and permanence; implementability; cost; and overall protection of human health and the environment. Comparisons are based on guidance provided in a July 24, 1987 EPA memo from J. Winston Porter and are first presented in detailed narrative discussion, and summarized by a check ("."), check-plus ("+"), check-minus ("-") scale. More detailed cost comparisons are then made, with sensitivity analyses based on capital cost, O&M cost, and present worth discount rate.

6.1 Design of Alternatives

A conceptual design has been developed for Alternatives C and D based on the GWRI, RA and appended information. These designs incorporate engineering judgment, vendor data, and experiences with comparable ground water remediation projects.

This section covers the design of a slurry wall and ground water recovery and treatment systems. The ground water treatment system and the recovery well construction would be common to both Alternatives C and D.

6.1.1 Design Basis

Ground water would be recovered only from the unconfined aquifer since the ground water analytical data indicate that no waste constituents are present in lower water bearing strata. Furthermore, the Stratum D aquifer below this upper aquifer is confined and for most of the year is under a higher hydraulic head; thus if these two aquifers were interconnected, flow would be upward into the unconfined aquifer, as stated in the GWRI.

The following common design basis is used for both Alternatives C and D. The data for this basis is taken from the ground water remedial investigation and the Source Control Remedial Investigation.

The normal volume of ground water recovered would be approximately twenty gallons per minute based on the unconfined aquifer pump test (Source Control RI). See Appendix B for supporting calculations. The ground water treatment is based on the design influent analysis (Table 6-1).

The influent analysis has been selected based on the highest detected level in any well for each component. It is very conservative as it does not take credit for dilution from wells which are outside the plume of waste constituents. The influent analysis upon which the preliminary design is based includes only those constituents detected in the ground water.

The recovery wells or slurry wall placement would be based on intercepting a 1500 feet long section of the site. Well spacing would need to be one every twenty feet based on the hydraulic conductivity data obtained in the unconfined aquifer pumping test.

6.1.2. Common Design Elements

Ground Water Treatment Unit

The ground water treatment unit would be common to both alternatives C and D. It is therefore described here and not duplicated under the design sections for both alternatives.

The proposed process is described as follows. Water from the recovery pumps located in the wells would be discharged into a holding tank designed for twelve hours retention time to equalize the influent flow to the unit. The influent would be filtered through sand and cartridge type filters, and treated through two

Table 6-1

Design Influent Concentration

Constituent	Concentration mg/l [a]
trans-1,2 dichloroethylene	0.025
Trichloroethylene	0.015
Tetrachloroethylene	0.021
Benzene	0.027

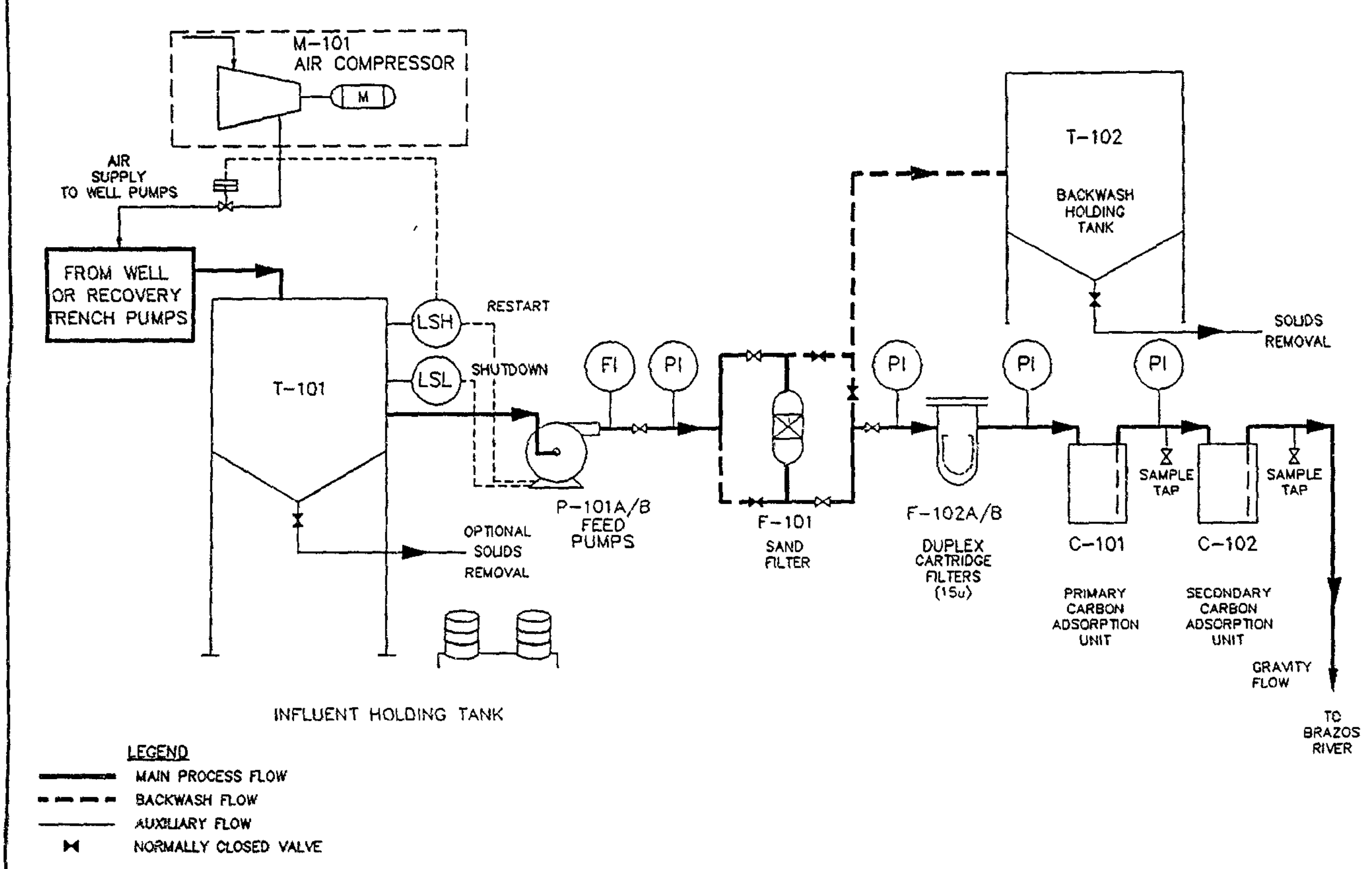
014670

[a] Based on the highest level detected in one of the wells MW-34, MW-37, or MW-38 during the 10/29/87 sampling event.

granular activated carbon (GAC) adsorption vessels piped in series. The effluent of the primary adsorber would be monitored for break-through of any of the four constituents listed in Table 6-1 and would be replaced when breakthrough occurs. The secondary unit would be moved into the primary position and a new secondary unit would be placed. In this manner, a polishing unit would always be available to ensure that waste constituents are not present in the effluent. The polished effluent would flow by gravity to the Brazos River. Spent adsorber units would be shipped off-site for disposal or regeneration. Figure 6-1 depicts the proposed flow scheme of the treatment unit.

The design basis for the treatment unit is as follows:

Design Flow Rate	20 gpm normal, based on 7.1 gpm ground water discharge to river calculated during pumping test.
Influent Holding Tank	Twelve hours capacity at normal flow; 10,000 gallon capacity FRP tank.
Sand Filters	Removes larger particles of silt to decrease frequency of changes of the cartridge filters.
Cartridge Filters	Remove silt and particles down to 15 microns to protect the GAC unit from fouling with solids.
GAC Units	Sized for 11 minutes contact time in each vessel. Removes the constituents shown in Table 6-1 approximately to below detection limits. Carbtrol model L-1 or equivalent with 200 pounds of activated carbon in a 55 gallon steel drum.
Design GAC Consumption Rate	0.07 lb/1000 gallon treated water
Unit Change Out Rate	Approximately four units per year



ERM-Southwest, inc.

NEW ORLEANS, LOUISIANA   HOUSTON, TEXAS

DATE 06/20/89	W.O.NO. 9121A003F89

FIGURE 6-1
PRELIMINARY FLOW SCHEMATIC
GROUND WATER TREATMENT UNIT
SHERIDAN DISPOSAL SERVICES
HEMPSTEAD, TEXAS

014672

Recovery Well Design

The recovery well construction is also common to both alternatives C and D. Well design is described here and not duplicated under the design sections for both alternatives.

Well Depth	60 feet average
Well Construction	4" diameter polyvinyl chloride (PVC) casing w/screened interval 35-55 feet.
Recovery Pumps	Pneumatic submersible type; Sized for one gpm each; With high and low level controls.

Institutional Controls

Institutional controls are included in Alternatives B, C, and D to preclude use of ground water during the remediation time. Since, as shown in Table 5-1, with either active restoration or natural attenuation the time to extract one pore volume is more than 25 years, some long-term provision for controlling use of the shallow aquifer is required. As previously stated, there are no current shallow ground water users downgradient of the site between the pond and the river. Therefore, institutional control would be used to restrict new well construction. Extraction of affected ground water would be prevented by the use of access and deed restrictions recorded in the county clerk's office, since the plume is entirely on SDS property.

6.1.3 Alternative A - No Action Alternative

Description:

Alternative A, No Action includes no capital improvements at the site or any maintenance or monitoring efforts.

Overall Concepts:

None	No additional capital improvements made at the site.

6.1.4 Alternative B - Natural Attenuation with Institutional Controls and Monitoring

Description:

Alternative B relies on lowering contaminant concentrations through physical, chemical, and biological processes. This alternative also includes monitoring to track the direction and rate of movement of the plume, as well as responsibility for maintaining effective, reliable institutional controls to prevent use of the contaminated ground water. Surface water monitoring, both upstream and downstream of the site, would supplement ground water monitoring to ensure that ARARs continue to be met.

Overall Concepts:

Institutional Controls	Deed restriction recorded in the county clerk's office to prevent future use of affected ground water.
Monitoring	The existing wells, plus any others installed during the source remedy would be used for monitoring. No additional capital improvements would be made to the site.

014674

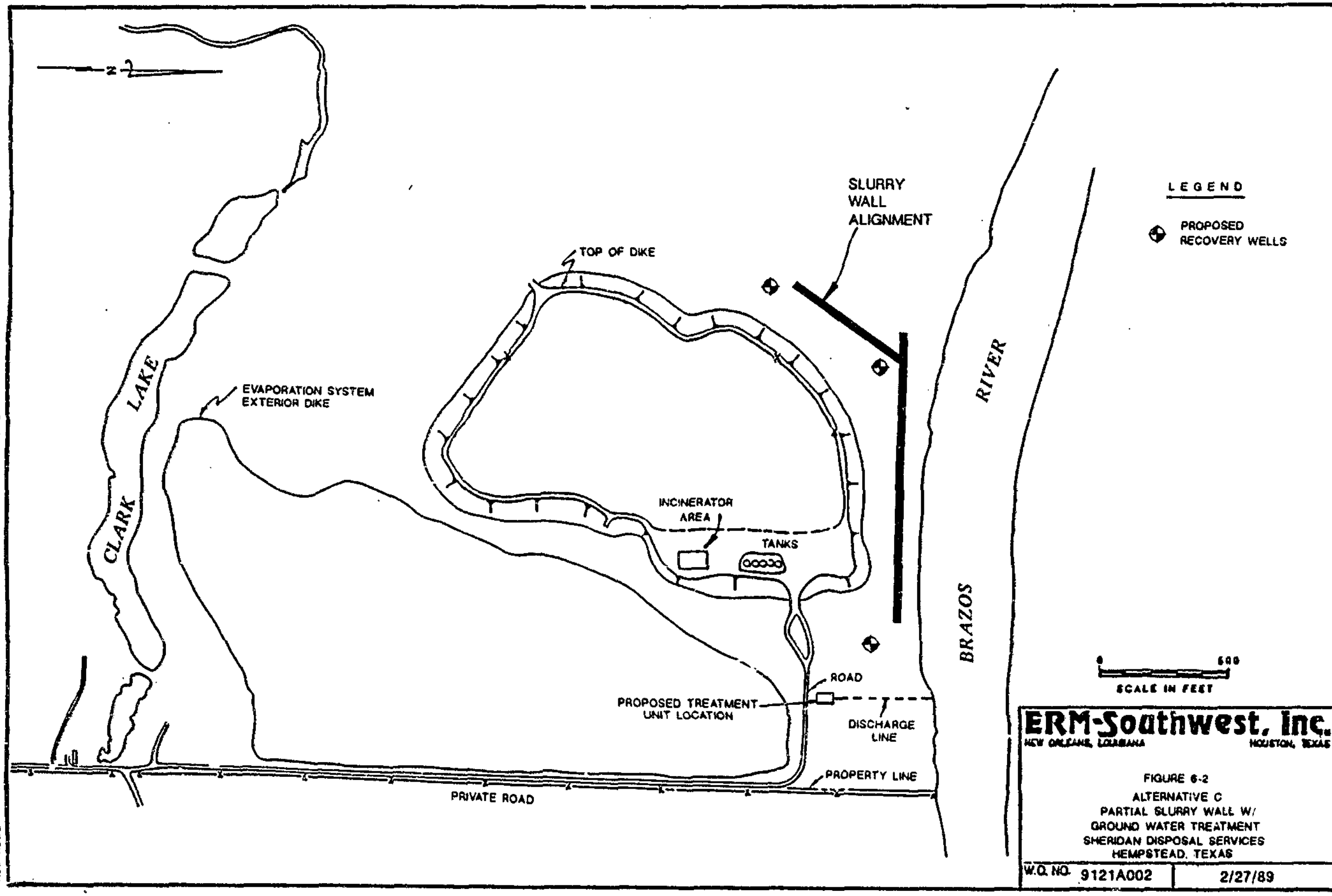
6.1.5 Alternative C - Partial Slurry Wall with Ground Water Treatment

Description:

A slurry wall located approximately as shown on Figure 6-2 will intercept the ground water flow and channel it to a group of wells located at each end of the impermeable wall. Ground water will be extracted with these wells and treated for removal of specific organic constituents. The treated effluent will be discharged to the Brazos River. When all wells are shut down, the treatment unit will be decontaminated and dismantled. It is estimated that the treatment unit will be operated for 25 years, based on modeling provided in Appendix A. At this rate, the ground water at the upgradient edge of the source area will reach the slurry wall in approximately 25 years.

Overall Concepts:

Institutional Controls	Deed restriction recorded in the county clerk's office to prevent future use of affected ground water.
Slurry Wall Length	1500 Feet


N
CLARK
LAKE
EVAPORATION SYSTEM
EXTERIOR DIKE
TOP OF DIKE
SLURRY
WALL
ALIGNMENT
LEGEND
PROPOSED
RECOVERY WELLS
RIVER
BRAZOS
INCINERATOR
AREA
TANKS
ROAD
PROPOSED TREATMENT
UNIT LOCATION
DISCHARGE
LINE
PROPERTY LINE
PRIVATE ROAD
0
500
SCALE IN FEET
ERM-Southwest, Inc.
NEW ORLEANS, LOUISIANA
HOUSTON, TEXAS
FIGURE 6-2
ALTERNATIVE C
PARTIAL SLURRY WALL W/
GROUND WATER TREATMENT
SHERIDAN DISPOSAL SERVICES
HEMPSTEAD, TEXAS
W.O. NO. 9121A002
2/27/89

Slurry Wall Depth	60 Feet minimum; keyed-in 3 feet to the aquitard below the unconfined aquifer (Stratum C)
Well Spacing and Number	3 wells located at each end and in the middle of the slurry wall.

Well construction and treatment unit are as described under Section 6.1.3, Common Design Elements.

6.1.6 Alternative D - Recovery Wells with Ground Water Treatment

Description:

This alternative consists of placing a line of wells located approximately as shown on Figure 6-3. Ground water will be extracted with these wells and treated for removal of specific organic constituents. The treated effluent will be discharged to the Brazos River. When all wells are shut down, the treatment unit will be decontaminated and dismantled. It is estimated that the treatment unit will be operated for 25 years, based on modeling provided in Appendix A. At this rate, the ground water at the upgradient edge of the source area will reach the intercepting line of wells.

014676

Overall Concepts:

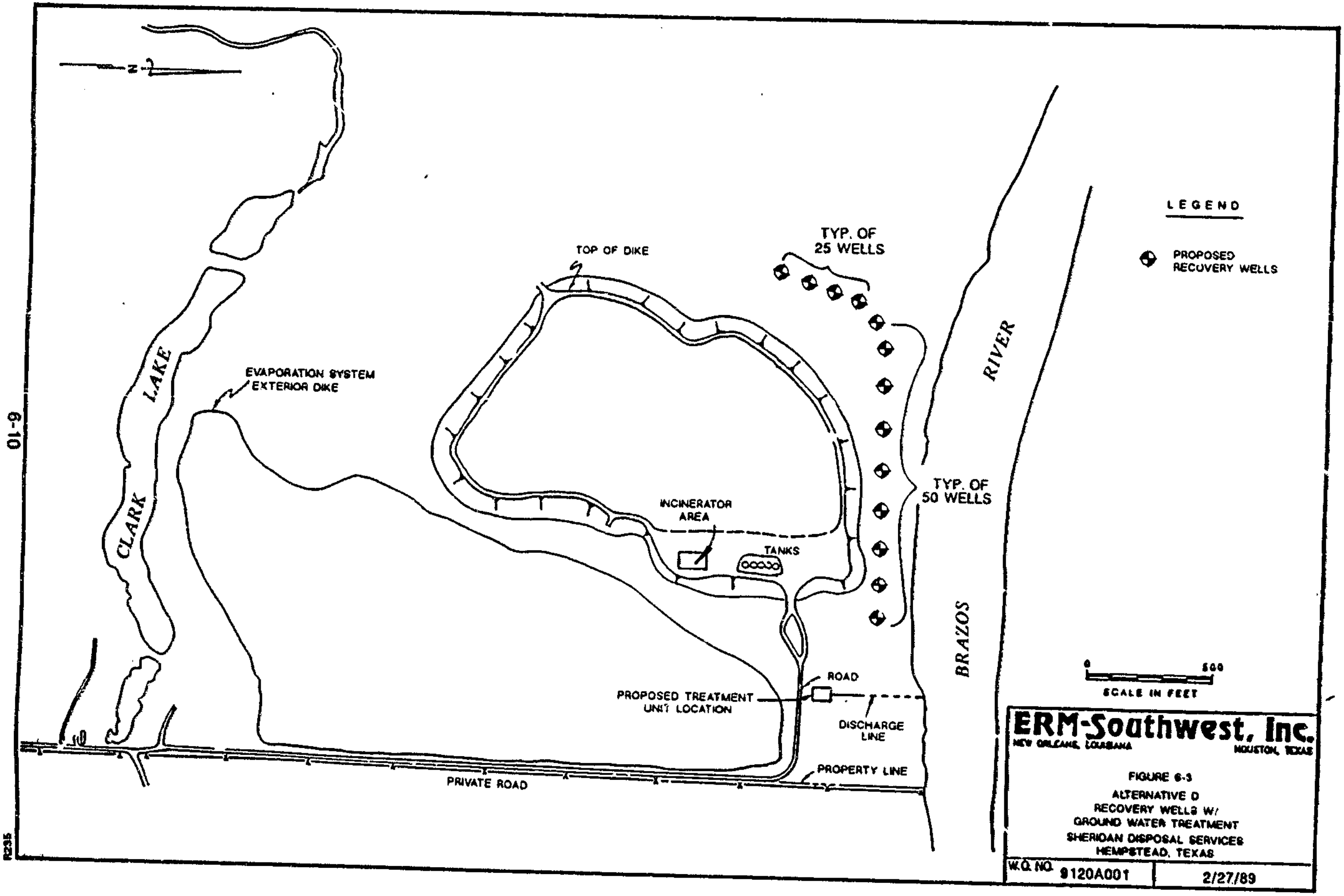
Institutional Controls	Deed restriction recorded in the county clerk's office to prevent future use of affected ground water.
Well spacing and number	75 wells on 20' centers

Well construction and treatment unit are as described under Section 6.1.3, Common Design Elements.

6.2 Comparative Evaluation of Alternatives

6.2.1 Comparative Evaluation Criteria

Having defined the three surviving alternatives in more detail, this section of the GWFS subjects each alternative to a comparative evaluation. This comparative evaluation is conducted on the basis of seven factors or criteria. These criteria include: (1) Compliance with ARARs; (2) Reduction in mobility, toxicity or volume; (3) Short term effectiveness; (4) Long term effectiveness and permanence; (5) Implementability; (6) Cost; and (7) Overall protection of human health and the environment.



ERM-Southwest, Inc.
NEW ORLEANS, LOUISIANA HOUSTON, TEXAS

FIGURE 6-3
ALTERNATIVE D
RECOVERY WELLS W/
GROUND WATER TREATMENT
SHERIDAN DISPOSAL SERVICES
HEMPSTEAD, TEXAS

W.O. NO. 9120A001 | 2/27/89

014677

The considerations relevant to the comparative evaluation for each of these seven criteria are outlined below, followed by the detailed evaluation of the relative strengths and weaknesses of the various alternatives on the basis of these considerations.

1. <u>Compliance with ARARs</u>

In determining appropriate remedial actions at Superfund sites, consideration is given to the requirements of other Federal and State environmental laws, in addition to CERCLA as amended by SARA. Primary consideration is given to attaining applicable or relevant and appropriate Federal and State public health and environmental laws and regulations and standards. Not all Federal and State environmental laws and regulations are applicable to each Superfund response action. Section 3 describes those ARARs specific to the Sheridan site. Section 6.2.3 evaluates the degree to which the selected alternates comply with these ARARs.

2. <u>Reduction in Toxicity, Mobility or Volume</u>

The degree to which alternatives employ treatment that reduces toxicity, mobility or volume is assessed. Relevant factors to this consideration include:

- o The treatment processes which the proposed solutions employ and materials they treat;
- o the amount of contaminated materials that will be destroyed or treated;
- o the degree of expected reduction in toxicity, mobility or volume;
- o the degree to which the treatment is reversible; and
- o the residuals that will remain following treatment, considering the persistence, toxicity, mobility, and propensity for bio-accumulation of such hazardous substances and their constituents.

3. <u>Short-term Effectiveness</u>

The short-term effectiveness of an alternative is assessed including a consideration of the following:

- Magnitude of reduction of existing risks; and

- short-term risks that might be posed to the community, workers, or the environment during the implementation of an alternative including potential threats to human health or the environment associated with excavation, transportation, or redisposal or containment.

4. <u>Long-term Effectiveness and Permanence</u>

Each alternative is assessed for the long-term effectiveness and permanence it affords along with the degree of certainty that the remedy will prove successful. Factors considered include:

- magnitude of residual risks in terms of amounts and concentrations of wastes remaining following implementation of a remedial action, considering the persistence, toxicity, mobility and propensity for bio-accumulation of such hazardous substances and their constituents;

- type and degree of long-term management required, including monitoring and operation and maintenance;

- potential for exposure of human and environmental receptors to remaining waste considering the potential threat to human health and the environment associated with excavation, transportation, redisposal, or containment;

- long-term reliability of the engineering and institutional controls, including uncertainties associated with the land disposal of untreated wastes and residuals; and

- potential need for replacement of the remedy.

5. <u>Implementability</u>

The ease or difficulty of implementing the alternatives is assessed by considering the following factors:

- degree of difficulty associated with constructing and maintaining the solution;

- expected operational reliability of the treatment technology;
- need to coordinate with and obtain necessary approvals and permits (or meet the intent of any permit in the case of Superfund actions);
- availability of necessary equipment and specialists; and
- available capacity and location of needed treatment, storage and disposal services.

6. <u>Costs</u>

The types of costs assessed include the following:

- capital costs;
- operation and maintenance costs;
- net present value of capital and operation and maintenance cost; and
- potential future remedial action costs.

7. <u>Overall Protection of Human Health and the Environment</u>

Following the analysis of the remedial options against individual evaluation criteria, the alternatives are assessed from the standpoint of whether they provide adequate protection of human health and the environment.

SARA directs EPA to give preference to solutions that utilize treatment to remove contaminants from the environment. Off-site transport and disposal without treatment is the least preferred option where practicable treatment technologies are available.

6.2.2 Evaluation Summary

The following values were assigned to compare remedial selection criteria:

"+" Alternative should exceed a criterion in comparison to other alternatives.

"." Alternative should meet the selection criterion.

"-" Alternative will not meet a criterion, or will not meet a criterion as well as other alternatives.

The rationale for the ratings assigned each alternative is presented in the following subsections.

6.2.3 Compliance with ARARs

The No Action Alternative is accorded a rating of "-" due to the inability to monitor the ground water and determine whether ARARs are continuing to be met for the long term. The Alternatives B, C, and D all meet ARARs and are rated ".".

6.2.4 Reduction of Toxicity, Mobility or Volume

Natural attenuation has some effect on the reduction of toxicity, mobility or volume of waste constituents given the site characteristics as stated in Section 4.2.3. For this reason, Alternatives A and B are ranked ".". Alternatives C and D include treatment and thus reduce the toxicity of the ground water. These alternatives are given a rating of "+". It should be noted that at the design flow rate and composition of the treatment scheme proposed for Alternatives C and D, less than eight pounds total of the four target organics would be removed in the first year and this quantity would very likely continue to decrease with time.

6.2.5 Short-Term Effectiveness

The No Action Alternative is ranked "-" due to the inability to prevent ground water use before attenuation takes place. The Natural Attenuation Alternative, for the short-term, is equally effective as Alternatives C and D. All three alternatives will result in ground water concentrations which meet ARARs and do not pose a risk to human health and the environment, as stated in Section 3. For this reason, Alternative B is ranked ".". Alternatives C and D will cause on-site workers to be exposed to some risk since these alternatives include active construction and operation activities. Therefore, Alternatives C and D should be ranked "-".

6.2.6 Long-Term Effectiveness and Permanence

The No Action Alternative is ranked "-" due to the inability to monitor whether ARARs are continuing to be met for the long term. In the long-term, the concentrations of constituents will be reduced by natural processes, therefore Alternative B is accorded a ranking of ".". Alternatives C and D will be slightly more effective at reducing the concentrations of constituents in the long-term. Both C and D are rated "+". It should be noted that alternatives C and D have the risks of extracting the ground water so that human contact may be made before either dilution or treatment, and concentrating them on GAC in a form that must be disposed of or regenerated.

6.2.7 Implementability

Alternative A and B would be the most easily implemented and are therefore rated "+". Between the remaining alternatives, Alternative D is more easily implemented than C. It is rated ".", since it requires construction of wells and a treatment plant. Alternative C, partial slurry wall with ground water treatment, is rated "-" due to the difficulties in constructing a slurry wall considering the site constraints. Site constraints include a narrow strip of land for access, the fact that a trench of 65' depth is beyond the reach of normal trenching equipment and a new working "bench" must be constructed.

6.2.8 Cost

Table 6-2 summarizes the total cost of the alternatives as developed in detail in Section 6.3 and in Appendix C. Costs are presented as capital, operating and maintenance and total cost. The No Action and Natural Attenuation with Institutional Controls and Monitoring alternatives are the least costly alternatives and are both ranked "+". Alternative C is second in terms of cost and is rated ".". Alternative D is the most costly alternative and is therefore rated "-".

6.2.9 Overall Protection of Human Health, Environment

The No Action Alternative is ranked "-", due to the inability to prevent potential use of affected ground water and lack of monitoring. Alternative B is ranked "." since the seepage of ground water into the Brazos River under current and projected future conditions will result in concentration levels which are protective of human health and the environment. Institutional controls would effectively prevent use of the affected ground water. Alternatives C and D are equivalent to Alternative B in terms of overall protection of human health and the environment and are therefore rated "." The reasons for this ranking are discussed below:

Table 6-2

Present Value Cost Summary For Alternatives

Cost Item	Alt. A - No Action ($M)	Alt. B - Natural Attenuation w/Institutional Controls and Monitoring ($M)	Alt. C - Partial Slurry Wall w/ Ground Water Treatment ($M)	Alt. D - Recovery Wells s/Ground Water Treatment ($M)
Estimated Capital Cost	0	0	850	1,095
Total Operating Cost [a]	0	326	4,106	5,171
PV Operating Costs [b]	0	194	1,744	2,180
Total Alternative Cost [a]	0	326	4,956	6,266
PV Alternative Cost [b]	0	194	2,594	3,275

[a] This cost represents present value of 30 years annual costs assuming that the effect of inflation cancels the after-tax rate.

[b] 30 Year present value with annual interest rate = 5% and inflation = 0%

The shallow ground water recovery rate is relatively low, therefore withdrawal of one pore volume will require greater than 25 years. Since extraction of several pore volumes is frequently necessary in similar situations to achieve the remedial objectives, it is anticipated that treatment would continue for some multiple of 25 years. During this relatively long time period, the shallow ground water would not meet drinking water criteria and could not be used as such. Institutional controls would be maintained for this period to prevent potable use of the shallow aquifer. The reality of the situation under any of Alternatives B, C or D is thus identical, i.e., long-term institutional controls are required to prevent use of the shallow aquifer.

Furthermore, although Alternatives C and D would effect a reduction in toxicity, mobility or volume as stated in Section 6.2.4, this reduction is minimal. At the design flow rate and composition of the proposed treatment scheme, less than eight pounds total of the four target organics would be removed in the first year and this quantity would likely decrease with time. This small reduction in waste constitutents is obtained by concentrating the constituents on GAC which will then have to be regenerated or disposed of.

6.2.10 Summary of Comparative Analysis

Table 6-3 presents a summary of the ranking of alternatives presented in this section. In terms of compliance with ARARs, all alternatives except No Action satisfy both the regulatory and the risk-based objectives. The Natural Attenuation with Institutional Controls and Monitoring alternative is fully protective of human health and the environment. The other alternatives, C and D, make a slight reduction of toxicity of the affected ground water, but the reduction is very small, and the decrease in surface water concentrations would not be detectible, as previously stated. Furthermore, these alternatives have the disadvantage of concentrating waste constituents on GAC, which must be disposed of or regenerated.

For the cost criterion, the ranking varied among the alternatives with cost generally increasing from Alternatives A through D (ranging from $194,000 for Natural Attenuation with Institutional Controls and Monitoring to $3,275,000 for recovery wells on a present value basis). The more costly alternatives (Alternatives C and D), are generally are more difficult to implement and they may pose more short-term risks to on-site workers. Further, Alternatives C and D will not appreciably decrease the time necessary to achieve remedial objectives.

Table 6-3

Summary Ranking of Alternatives

Alternative	Compliance With ARARs	Toxicity Mobility or Volume Reduction	Short-Term Effectiveness	Long-term Effectiveness, Permanence	Implementability	Cost	Overall Protection of Human Health, Environment
A - No Action	-	●	-	-	+	+	-
B - Natural Attenuation with Monitoring	●	●	●	●	+	+	●
C - Partial Slurry Wall w/ Ground Water Treatment	●	+	-	+	-	●	●
D - Recovery Wells w/Ground Water Treatment	●	+	-	+	●	-	●

NOTES:

"+" Alternative exceeds a criterion in comparison to other alternatives.
"●" Alternative meets the selection criterion.
"-" Alternative will not meet a criterion, or will not meet a criterion as well as other alternatives.

6.3 Cost

6.3.1 Total Cost

A cost was systematically estimated for each alternative from a foundation of common unit costs. Estimated costs were developed sequentially as follows:

1. Unit Costs - unit costs for remediation activities common in the region.

2. Options Costs - costs for treatment and containment options to be incorporated into assembled alternatives. Based on concept designs in Section 6.1.

3. Alternatives Costs - estimated total cost for each alternative. Based on concept designs in Section 6.1. Contains unit costs, derived unit costs and options costs. Includes contingencies, operating and post-closure monitoring costs.

Tables 6-4 through 6-6 summarize the estimated total cost for alternatives B, C, and D.

Appendix C contains back-up information and calculations used in the development of the estimated total cost for each alternative. This appendix includes a summary table of derived unit costs and assumed unit costs based on experience. Special concerns about present worth analysis of costs are discussed in the following paragraphs.

For this Feasibility Study the following present worth assumptions were used:

- o term = 25 years
- o interest rate = 0
- o inflation = after-tax interest rate

The 25 year term has been estimated as the travel time for ground water at the southern most edge of the site to reach the recovery system. Historically in this country inflation is approximately equal to interest paid on certificates of deposit after corporate taxes are deducted. A PRP group which funds the remediation and long-term maintenance of a Superfund site typically creates a sinking fund or trust fund at the beginning or end of site remediation. This sinking fund is typically invested in insured securities, and

Table 6-4
Estimated Total Cost
Alternative 8 - Natural Attenuation
With Institutional Controls and Monitoring

The estimated total cost is the sum of capital cost, the cost of 30 years of annual maintenance and seven monitoring events [1, 3, 6, 10, 15, 20 and 30 years after closure]. All costs are 1989 costs. "Total" costs assume present value with interest rate cancelling the effect of inflation. "Present value" costs assume the interest rate = 5% and inflation = 0%.

	Quantity	Units	Unit Cost	Cost	Notes
Captial Cost	-	-	-	0	Assume wells installed during Source Remediation.
30 Year Monitoring	7	Events	$33,000	$231,000	Table C-1
Inspection/Well Maintenance	30	Events	1,000	30,000	Allowance
Subtotal				$261,000	
Contingency				X 1.25	
Total 30 Year Operating Costs				$326,000	(Rounded)
Present Value - 30 Year Operating Costs				194,000	
Total Estimated Alternative Cost				326,000	
Present Value Estimated Alternative Cost				194,000	

Table 6-5
Estimated Total Cost

Alternative C - Partial Slurry Wall w/ Ground Water Treatment

The estimated total cost is the sum of capital costs, the cost of 25 years of annual maintenance and seven monitoring events (1, 3, 6, 10, 15, 20 and 25 years after closure). All costs are 1989 costs. "Total" costs assume present value with interest rate cancelling the effect of inflation. "Present value" costs assume the interest rate = 5% and inflation = 0%.

	Quantity	Units	Unit Cost	Cost	Notes
Capital Cost (Rounded to 1000's)					
Mobilization/Demobilization	1	L.S.	$40,000	$ 40,000	Allowance
Slurry Wall Construction (1500' length X 65' Depth)	97,500	Sq.Ft.	2.50	244,000	Unit Cost quoted by Chris McGee of Geocon
Well Installation	3	Each	4,000	12,000	Table C-2
Treatment Unit	1	L.S.	158,000	158,000	Table C-3
Well Pumps Installation	1	L.S.	39,000	39,000	Table C-4
Subtotal				493,000	
Contractor Overhead, Profit, Bond,				X 1.15	
Engineering & Construction Surveillance				X 1.20	
Contingency				X 1.25	
Estimated Capital Cost				850,000	(Rounded)
25 Year Operating Cost					
Ground Water Monitoring (25 yrs)				157,000	
Treatment Unit Operation	25	Years	$91,000	2,275,000	Table C-5
Well Pumps Maintenance	25	Years	1950	49,000	5% of Capital Cost Annually
Well Pumps Operation	2,450,000	KWh	0.08	196,000	Cost of running 15 hp air compressor continuously for 25 years
Subtotal				2,677,000	
Contingency				X 1.25	
Total Operating Costs				3,346,000	(Rounded)
Present Value Operating Costs				1,578,000	
Total Estimated Alternative Costs				4,196,000	
Present Value Estimated Alternative Cost				2,428,000	

1921

Table 6-6
Estimated Total Cost
Alternative D - Recovery Wells with Ground Water Treatment

The estimated total cost is the sum of capital cost, the cost of 25 years of annual maintenance and seven monitoring events [1, 3, 6, 10, 15, 20 and 25 years after closure]. All costs are 1989 costs. "Total" costs assume present value with the interest rate cancelling the effect of inflation. "Present value" costs assume the interest rate = 5% and inflation = 0%.

	Quantity	Units	Unit Cost	Cost	Notes
Capital Cost (Rounded to 1000's)					
Mobilization/Demobilization	1	L.S.	$40,000	$ 40,000	Allowance
Install Wells	75	Each	4,000	300,000	Table C-2
Well Pumps Installed	1	L.S.	137,000	137,000	Table C-6
Treatment Unit	1	L.S.	158,000	158,000	Table C-3
Subtotal				$ 635,000	
Contractor Overhead, Profit, Bonds				X 1.15	
Engineering and Construction				X 1.20	
Surveillance Contingency				X 1.25	
Estimated Capital Cost				$1,095,000	(Rounded)
Operating Cost					
Ground Water Monitoring				$ 157,000	
Treatment Unit Operation	25	Years	$91,000	2,275,000	Table C-5
Well Pumps Maintenance	25	Years	6,850	171,000	5% of Capital Cost Annually
Well Pumps Operation	9,800,000	KWh	0.08	784,000	Cost of power for 60 hp air compressor running continuously for 25 years
Subtotal				3,387,000	
Contingency				X 1.25	
Total Operating Costs				4,234,000	(Rounded)
Present Value Operating Costs				1,978,000	
Total Estimated Alternative Cost				5,329,000	
Present Value Estimated Alternative Cost				3,073,000	

014689

is calculated to be sufficient to pay for the annual O&M costs for a designated period of time. Since a PRP group can not have non-profit status under the current tax law, it must pay taxes on the interest earned.

Since inflation is a very real economic phenomenon, a PRP group must set aside funds to provide for future increases in annual O&M costs. Historically, interest on invested securities is typically greater than inflation by one-third to one-half. Current corporate tax rates are 34% with a current surcharge of 5%. These taxes are either paid by each member company or by the PRP group directly. After taxes are deducted from interest earned, the net interest earned on the invested funds approximately offsets the increased annual costs due to inflation. On this basis, i = 0 in the present worth formula. Following these assumptions, the 25-year present worth of an O & M cost of $1/year is $25.

Alternately, if one were to use the Federal government's guidelines for calculating present value for 25 years using 5% interest and 0% inflation, the present worth of a $1/year expenditure for 25 years is $14.09. As this illustrates, neglecting inflation will cause annual O & M costs to be understated, possibly resulting in the selection of a remedial plan that has lower capital or first year costs and higher annual or reoccurring O & M costs.

For clarity, the cost calculations and summaries are presented both ways.

6.3.2 Sensitivity Analysis

Capital Cost Sensitivity Analysis

The cost estimates presented in the FS are to represent +50%/-33% accuracy, so these are the sensitivity limits chosen. Table 6-7 shows that the relative cost rankings are not altered by the changes in capital costs. There is very little sensitivity in total cost to changes in capital cost.

O & M Cost Sensitivity Analysis

The costs were varied within the same +50%/-33% estimate accuracy range as the capital costs. Again, the relative cost rankings are not changed by changing the O & M costs. Table 6-8 reflects more total cost sensitivity to O&M cost variability than to capital cost since O&M represents a larger fraction of the overall cost. However, the differences in cost between alternatives are not significant.

1922

Table 6-7

Capital Cost Sensitivity Analysis

Table 6-2 Cost Change [a]	Alt. B - Natural Attenuation ($M)	Alt. C - Partial Slurry Wall w/Ground Water Treatment ($M)	Alt. D - Recovery Wells w/Ground Water Treatment ($M)
Total [b]	326	4,196	5,329
50%	326	4,621	5,876
28%	326	4,434	5,635
0%	326	4,196	5,329
-20%	326	4,026	5,110
-33%	326	3,916	4,967

[a] The Table 6-2 Total Alternative Cost with the indicated percent change change in capital costs.

[b] After tax I • inflation.

014691

1923

Table 6-8

O & M Cost Sensitivity Analysis

Table 6-2 Cost Change [a]	Alt. B - Natural Attenuation ($M)	Alt. C - Partial Slurry Wall w/Ground Water Treatment ($M)	Alt. D - Recovery Wells w/Ground Water Treatment ($M)
Total [b]	326	4,196	5,329
50%	489	5,870	7,446
28%	418	5,133	6,514
0%	326	4,196	5,329
-20%	261	3,527	4,482
-33%	218	3,092	3,932

[a] The Table 6-2 Total Alternative Cost with indicated percent change in Operating & Maintenance Costs

[b] After tax i = inflation

014692

Discount Rate Sensitivity Analysis

The discount rate utilized for the present worth calculations was varied in a range from 3% to 10%. Table 6-9 reflects the fact that the total cost is not sensitive to this range of discount rates.

Volume of Water Treated Sensitivity Analysis

The sensitivity of cost to changes in the volume of treated water is presented in Table 6-10. Costs are presented for variations in both the flow rate and the time that the treatment unit would be operated. The capital cost of the treatment unit is scaled from the original or base case cost estimated (Table C-3) by using the "six-tenths"-factor rule, i.e.

$$\frac{\text{Cost A}}{\text{Cost B}} = \left(\frac{\text{Capacity A}}{\text{Capacity B}}\right)^{0.6}$$

(Reference: Peters and Timmerhaus, Plant Design and Economics for Chemical Engineers, p.107.) The O&M costs were split into categories of fixed and variable costs and recalculated for each case. The detailed calculation of these costs is presented in Appendix C. As Table 6-10 shows, the cost of alternative B does not change while the cost for Alternative C and D vary significantly with volume. However, the magnitude of change is comparable for both C and D, therefore variations in the volume of water treated would not change the relative rankings on the basis of cost.

014693

1924

Table 6-9

Present Worth Discount Rate Sensitivity Analysis

Discount Rate [a] %	Alt. B - Natural Attenuation ($M)	Alt. C - Partial Slurry Wall w/Ground Water Treatment ($M)	Alt. D - Recovery Wells w/Ground Water Treatment ($M)
Total [b]	194	2,428	3,073
3%	232	2,762	3,502
4%	212	2,582	3,270
5%	194	2,428	3,073
6%	179	2,296	2,904
7%	166	2,182	2,758
8%	155	2,083	2,631
9%	145	1,997	2,521
10%	137	1,922	2,425

[a] The indicated discount rate is used to calculate Present Value Estimated Alternative Cost for each alternative from Table 6-2.

[b] Present Value assuming i = 5% as given in Table 6-2.

014694

L556

Table 6-10
Volume of Water Treated
Sensitivity Analysis

Basis of Case	Total Volume of Water Treated (MM Gallons)	Alternative B Natural Attenuation ($M)	Alternative C Partial Slurry Wall w/ Ground Water Treatment ($M)	Alternative D Recovery Wells w/ Ground Water Treatment ($M)
10 gpm for 25 years	158	$326	$3,374	$4,139
10 gpm for 75 years	473	326	8,606	10,411
20 gpm for 25 years	315	326	4,197	5,330
20 gpm for 75 years	946	326	10,892	13,799

APPENDIX A

GROUND WATER RECOVERY MODELING

014696

L880

L-0705RI

JACOBS ENGINEERING GROUP INC.

ENVIRONMENTAL SYSTEMS DIVISION

12600 WEST COLFAX AVENUE SUITE A300, LAKEWOOD, COLORADO 80215
TELEPHONE (303) 232-7093

Ms. Ruth Izraeli, 6HET
U.S. EPA, Region VI
1445 Ross Aveue at Fountain Place
Dallas, Texas 75202-2733

Subject: Sheridan Disposal Service
Groundwater Migration Management FS
Supplemental Analyses of
Extraction Well Field and
Slurry Wall Efficiency

014697

Dear Ruth:

As requested, we have performed supplemental analyses, using the RESSQ model, of the efficiency of proposed well-field and slurry wall alignments in controlling the capture and rate of recovery of groundwater contaminants from the shallow aquifer beneath the Sheridan disposal impoundement.

Attachment 1 describes the model, the assumptions used, and the results of this brief modeling effort. In general, the conclusions show that a line of sufficiently closely-spaced groundwater wells on the downgradient side of the impoundment could be effective in capturing contaminants migrating with the local groundwater flow. The use of production wells would also accelerate slightly the rate of groundwater migration beneath the impoundment. However, for the well-field alignment considered, the changes in groundwater flow rates induced by production wells would be comparable to natural variations in flow rates observed in recent years at the site. To this extent, then, it can be concluded that the use of groundwater production wells would not substantially accelerate the rate of recovery of contaminated groundwater.

An attempt was also made to consider the effects of a partial slurry wall with extraction wells. Since the RESSQ model is analytical rather than numerical, it can not directly incorporate an imbedded zero-permeability zone. However, the slurry wall was treated as a no-flow boundary using the method of images to assess the relative effectiveness of the slurry wall with extraction wells versus the line of extraction wells (Attachment 1). The results suggest that the two options would be comparable in their effects.

L-0705RI

Ruth Izraeli
July 5, 1989

For your information, I have included an extract from the publication by Javandel et al. (Groundwater Transport: Handbook of Mathematical Models, American Geophysical Union Water Resources Monograph 10, 1984) which describes data input to the RESSQ model, as well as copies on diskette of the RESSQ software distributed by the International Ground Water Modeling Center.

With regard to sorption coefficients of benzene and TCE, a common approach is to utilize published relationships between the organic carbon content of aquifer materials and the organic carbon or octanol-water partition coefficients for the contaminants, as discussed in Attachment 2. For site data, estimated retardation coefficients for benzene may be in the range of 1 to 26, and for TCE in the range of 1 to 37.

014698

I trust that this information will be of use to you in finalizing the Sheridan Ground Water Migration Management FS. If you have any questions, please call me.

Yours truly,
JACOBS ENGINEERING GROUP INC.

Donald W. Beaver, Ph.D.
Senior Geohydrologist

Attachments

cc: Al Medine
Linda Chapman

Attachment 1
RESSQ Model Formulation and Results

Introduction

The RESSQ model is a semi-analytical model of two-dimensional contaminant transport by advection and adsorption in a homogeneous, isotropic aquifer of uniform thickness. It is based on the assumption that uniform regional flow, sources, and sinks create a steady-state (or nearly so) flow field in the aquifer. RESSQ calculates streamline patterns, locations of contaminant fronts at various times, and variations of contaminant concentrations with time at sinks. Only some of the capabilities of RESSQ have been employed in the present effort. Specifically, the model has only been used to calculate streamlines and contaminant capture times. Concentration distributions have not been considered, nor have the effects of sorption been considered.

The version of RESSQ used here was obtained from the International Ground Water Modeling Center (IGWMC) of the Holcomb Research Institute at Butler University. The software distributed by IGWMC is based on the model described by Javandel, Doughty, and Tsang (Groundwater Transport: Handbook of Mathematical Models, American Geophysical Union Water Resources Monograph 10, 1984). The software package distributed by IGWMC includes RESSQPLT, a post-processor which enables efficient screen graphics and plotting of model output.

Model Input

In addition to various control parameters, input to RESSQ includes the following:

- Numbers of injection and production wells
- Ambient contaminant concentration in aquifer
- Contaminant concentration at injection wells
- Aquifer thickness and porosity
- Rate and direction of regional average pore velocity
- Retardation coefficient
- Locations and rates of injection and extraction wells

Input can be in the cgs system of units, or in a "practical" system of metric units, in which distance is given in meters, regional flow velocity in meters per year, and well flow rates in cubic meters per hour. For further discussion of model input, see Javandel et al. (op. cit.).

Line of Injection Wells

Input Data. Input data for the RESSQ simulations were derived from various sections of the Ground Water Migration Management FS. Subsequent references to page, table, figure and appendix numbers are to that document. Input data included geometry of the assumed contaminant source area, average regional pore velocity, length of the line of production wells, and total production from the wells. The following data were used:

- Aquifer thickness (b) = 7.3 m (Table 3-3)
- Porosity (n) = 0.3 (page 2-9)
- Plume width (W) = 284.7 m or 934 ft (Table 3-3)
- Plume area (A) = 1,674,444 ft^2 (Page 2-9)
- Hydraulic conductivity (K) = 6.83 m/dy (Table 3-3)
- Gradient (i) = 0.0023 (Table 3-3)

The average regional pore velocity (v) was calculated as

$$v = Ki/n = 19 \text{ m/yr}$$

The line of downgradient wells was distributed along a distance of 290 meters, slightly greater than the reported plume width of 284.7 meters. The production rate per well was estimated based on the downgradient line of 50 wells (Figure 6-3, neglecting the line of 25 wells to the northwest of the impoundment), each producing at a rate of 0.24 gpm (Appendix B), for a total production of 12 gpm. Since the RESSQ software permits a maximum of only thirty production wells to be simulated without editing and recompiling, it was assumed that 30 production wells were distributed along the downgradient edge of the impoundment, each producing 0.4 gpm (0.0909 m^3/hr) and spaced 10 m apart. This approach preserves the total production and total length of the line of extraction wells, although the wells are spaced slightly farther apart and assumed to produce at a slightly higher rate than as indicated in Appendix B.

The average plume length was calculated as

$$L = A/W = 1,674,444 \text{ ft}^2/934 \text{ ft}$$

$$= 1793 \text{ ft} = 546 \text{ m}$$

Consequently, a line of zero-rate "injection wells" was placed 550 m upgradient from the line of extraction wells. Three of these points were placed near the centerline of the contaminant plume, and two were placed near the lateral margins of the plume. These zero-rate injection wells permit the tracing of streamlines from them to the production wells.

For baseline comparison purposes, the time of travel for a distance of 550 m at a rate of 19 m/yr is 28.95 years, which was generally rounded to 30 years in the FS as the approximate time for one pore volume of contaminants to pass from beneath the impoundment.

<u>Model Output</u>. Listings of the input, output, and graphical streamline output for the case of regional flow only and the line of production wells are given in Attachments 1A and 1B, respectively. For the line of production wells, the time required for capture of individual streamlines ranges from 21.5 to 22.4 years, about seven years faster than the time of travel under natural flow conditions only. (Note that to transport three pore volumes, as may be required for complete contaminant removal, the line of wells would require about 66 years, versus about 87 years under natural flow alone.) It should also be noted that the model limitation to 30 wells rather than 50, each producing at 0.4 gpm rather than 0.24 gpm, will accelerate the flow along each streamline at the end of the flow path, near each extraction well. Thus the calculated travel time of about 22 years is less than would be observed if more, lower-production rate extraction wells were simulated. Consequently, the actual difference in travel times between natural conditions and under the influence of a line of production wells would be less than seven years.

Note further that the model indicates that at least one streamline would not be captured by the simulated line of extraction wells. This supports the conclusion of Appendix B that more closely-spaced wells producing at lower rates would be required to effect complete recovery of contaminated groundwater from beneath the impoundment.

Partial Slurry Wall With Extraction Wells

The partial slurry wall consists of a no-flow boundary of finite length, oriented normal to the direction of regional flow. Such a boundary can be readily simulated by means of a numerical finite-difference or finite-element model, but not by an analytical model such as RESSQ. RESSQ can incorporate no-flow boundaries by means of image wells in the absence of uniform regional flow, and this approach has been applied here to obtain an indication of the possible effectiveness of a slurry wall with extraction wells. This indication is obtained by comparing the results of the line of extraction wells (in the absence of regional flow) with the results of three extraction wells and image wells located across the slurry wall alignment (in the absence of regional flow).

Input Data. Input data for the line of extraction wells are the same as previously discussed, except that the regional pore velocity is reduced to zero.

For the slurry wall with three extraction wells, the slurry wall is not directly simulated. Rather, three extraction wells are assumed to be located at the ends and in the center of the slurry wall alignment (which is taken equal to the alignment of the line of extraction wells considered previously). Total production from the three extraction wells is taken equal to the natural groundwater flow rate. Each of the extraction wells has an image well, producing at an equivalent rate, located across the assumed slurry wall alignment. Thus the slurry wall will be represented as a no-flow boundary by standard image-well theory.

The groundwater flow rate Q is given by

$$Q = KiA,$$

where A is the aquifer area normal to the groundwater flow direction. For this simulation, A is given by the product of the aquifer thickness b and the width L of the contaminant plume. Thus Q is 1.39 m^3/hr, or 0.463 m^3/hr for each of the three wells (and each of the three image wells).

Model Output. Listings of the input, output, and graphical streamline output for the case of the line of extraction wells and the partial slurry wall with extraction wells are given in Attachments 1C and 1D, respectively. For the line of extraction wells (with no regional groundwater flow), the travel time from the upgradient edge of the contaminant plume ranges from 97.7 to 102.0 years, with an average of 100.0 years for the five streamlines considered. For the partial slurry wall with extraction wells, the travel times for the five streamlines range from 101.9 to 107.2 years, with an average of 104.1 years. These results suggest that the line of extraction wells and partial slurry wall with extraction wells would have similar effectiveness in controlling contaminant migration, with the line of extraction wells being slightly more effective.

ATTACHMENT 1A

```
SHERIDAN - UNIFORM FLOW
   5     0 0.             0.
7.3         0.3          19.0        0.0         0.
  INWELL01      0.       135.         .0000                                      0.           1  1-1
  INWELL02      0.        15.         .0000                                      0.           1  1-1
  INWELL03      0.         0.         .0000                                      0.           1  1-1
  INWELL04      0.       -15.         .0000                                      0.           1  1-1
  INWELL05      0.      -135.         .0000                                      0.           1  1-1
   1        20.
50.                                -1
 -50.       600.        -160.       160.
```

014705

SHERIDAN - UNIFORM FLOW

PRACTICAL SYSTEM OF UNITS IS USED

REGIONAL FLOW, PORE VELOCITY	=	19.00 M/YR
ORIENTATION OF REGIONAL FLOW	=	0.00 DEGREES
THICKNESS OF THE AQUIFER	=	7.30 METERS
POROSITY	=	30.00 PERCENT
PERIOD STUDIED	=	50.00 YEARS
INITIAL AQUIFER CONCENTRATION	=	0.000E-01 PERCENT
DEFAULT INJECTION CONCENTRATION	=	0.000E-01 PERCENT
STREAMLINE STEP LENGTH	=	3.25 METERS
ABSORPTION CAPACITY OF ROCK	=	00.00 PERCENT

FRONTS ARE PLOTTED AT 20.000 YEARS

014706

5 INJECTION WELLS

WELL NAME	X METERS	Y METERS	FLOW-RATE M3/H	CONCENTRATION PERCENT	RADIUS METERS	INDICATOR
INWELL01	0.00	135.00	0.00	0.00E-01	7.50E-02	-1
INWELL02	0.00	15.00	0.00	0.00E-01	7.50E-02	-1
INWELL03	0.00	0.00	0.00	0.00E-01	7.50E-02	-1
INWELL04	0.00	-15.00	0.00	0.00E-01	7.50E-02	-1
INWELL05	0.00	-135.00	0.00	0.00E-01	7.50E-02	-1

STREAMLINES DEPARTING FROM INJECTION WELL INWELL01

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	+++none+++	50.1 YEARS	0.0

014708

STREAMLINES DEPARTING FROM INJECTION WELL INWELL02

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	+++none+++	50.1 YEARS	0.0

014709

STREAMLINES DEPARTING FROM INJECTION WELL INWELL03

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	+++none+++	50.1 YEARS	0.0

014710

STREAMLINES [illegible]

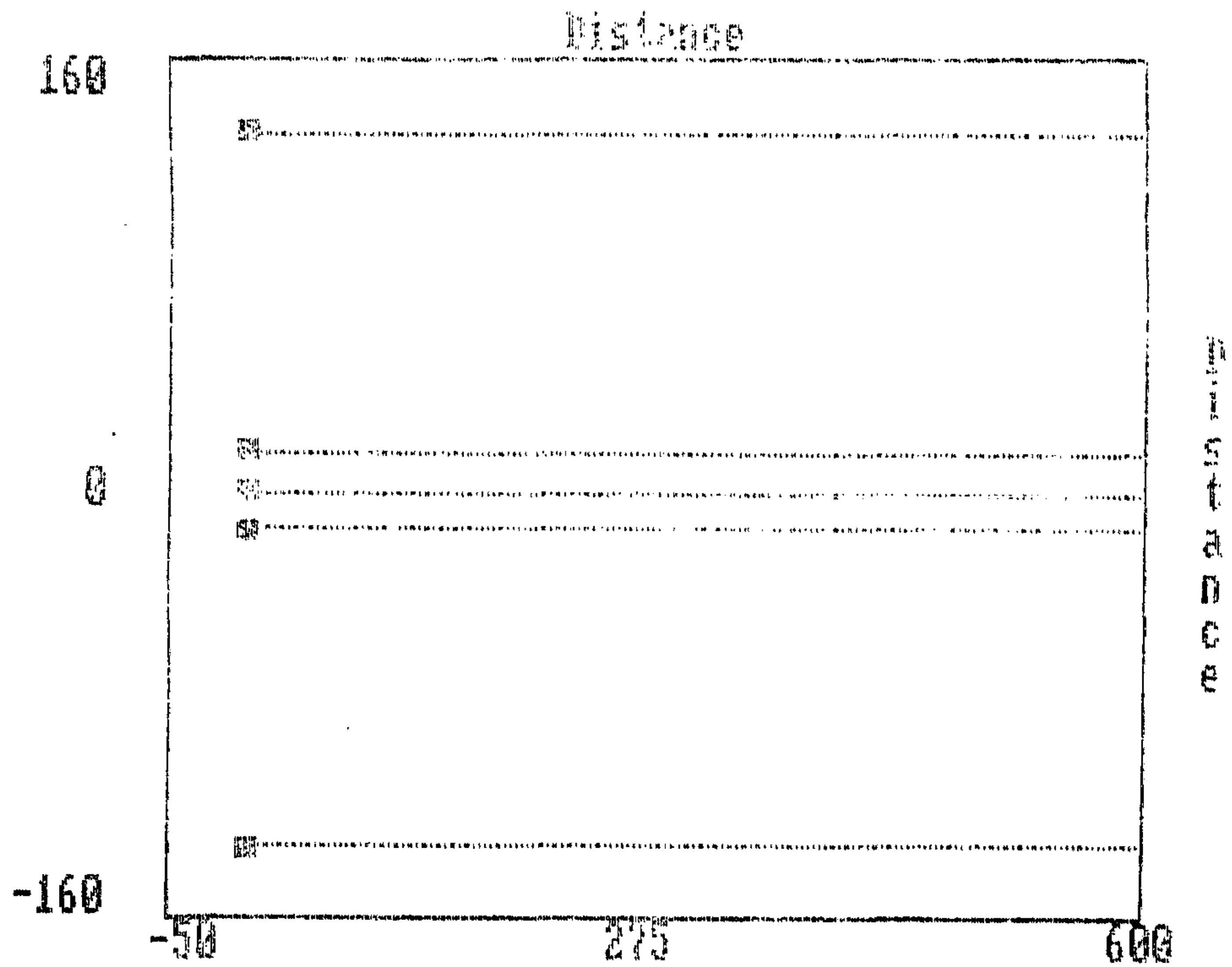
NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	+++none+++	50.1 YEARS	0.0

014711

STREAMLINES DEPARTING FROM INJECTION WELL INWELL05

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	+++none+++	50.1 YEARS	0.0

014712



014713

ATTACHMENT 1B

014714

MERIDIAN UNIFORM FLOW WITH LINE OF RECOVERY WELLS

5 30 C.		0.				
7 3	0.3	19.0	0.0	0.		
INWELL01	0.	135.	.000		0.	1 1-1
NWELL02	0.	15.	.000		0.	1 1-1
NWELL03	0.	0.	.000		0.	1 1-1
INWELL04	0.	-15.	.000		0.	1 1-1
NWELL05	0.	-135.	.000		0.	1 1-1
OUTWELL01	550.	5.	.0909			
UTWELL02	550.	-5.	.0909			
UTWELL03	550.	15.	.0909			
OUTWELL04	550.	-15.	.0909			
UTWELL05	550.	25.	.0909			
UTWELL06	550.	-25.	.0909			
OUTWELL07	550.	35.	.0909			
UTWELL08	550.	-35.	.0909			
UTWELL09	550.	45.	.0909			
OUTWELL10	550.	-45.	.0909			
UTWELL11	550.	55.	.0909			
UTWELL12	550.	-55.	.0909			
OUTWELL13	550.	65.	.0909			
ITWELL14	550.	-65.	.0909			
UTWELL15	550.	75.	.0909			
OUTWELL16	550.	-75.	.0909			
UTWELL17	550.	85.	.0909			
OUTWELL18	550.	-85.	.0909			
UTWELL19	550.	95.	.0909			
ITWELL20	550.	-95.	.0909			
OUTWELL21	550.	105.	.0909			
UTWELL22	550.	-105.	.0909			
UTWELL23	550.	115.	.0909			
OUTWELL24	550.	-115.	.0909			
UTWELL25	550.	125.	.0909			
UTWELL26	550.	-125.	.0909			
OUTWELL27	550.	135.	.0909			
ITWELL28	550.	-135.	.0909			
UTWELL29	550.	145.	.0909			
OUTWELL30	550.	-145.	.0909			
1	20.					
50.		-1				
- 50.	600.	- 160.	160.			

014715

PRACTICAL SYSTEM OF UNITS IS USED

REGIONAL FLOW, PORE VELOCITY	=	19.00	M/YR
ORIENTATION OF REGIONAL FLOW	=	0.00	DEGREES
THICKNESS OF THE AQUIFER	=	7.30	METERS
POROSITY	=	30.00	PERCENT
PERIOD STUDIED	=	50.00	YEARS
INITIAL AQUIFER CONCENTRATION	=	0.000E-01	PERCENT
DEFAULT INJECTION CONCENTRATION	=	0.000E-01	PERCENT
STREAMLINE STEP LENGTH	=	3.25	METERS
ADSORPTION CAPACITY OF ROCK	=	00.00	PERCENT

FRONTS ARE PLOTTED AT 20.000 YEARS

014716

5 INJECTION WELLS

WELL NAME	X METERS	Y METERS	FLOW-RATE M3/H	CONCENTRATION PERCENT	RADIUS METERS	INDICATOR
INWELL01	0.00	135.00	0.00	0.00E-01	7.50E-02	-1
INWELL02	0.00	15.00	0.00	0.00E-01	7.50E-02	-1
INWELL03	0.00	0.00	0.00	0.00E-01	7.50E-02	-1
INWELL04	0.00	-15.00	0.00	0.00E-01	7.50E-02	-1
INWELL05	0.00	-135.00	0.00	0.00E-01	7.50E-02	-1

30 PRODUCTION WELLS

WELL NAME	X METERS	Y METERS	FLOW-RATE M3/H	RADIUS METERS	INDICATOR
OUTWELL01	550.00	5.00	0.09	7.50E-02	0
OUTWELL02	550.00	-5.00	0.09	7.50E-02	0
OUTWELL03	550.00	15.00	0.09	7.50E-02	0
OUTWELL04	550.00	-15.00	0.09	7.50E-02	0
OUTWELL05	550.00	25.00	0.09	7.50E-02	0
OUTWELL06	550.00	-25.00	0.09	7.50E-02	0
OUTWELL07	550.00	35.00	0.09	7.50E-02	0
OUTWELL08	550.00	-35.00	0.09	7.50E-02	0
OUTWELL09	550.00	45.00	0.09	7.50E-02	0
OUTWELL10	550.00	-45.00	0.09	7.50E-02	0
OUTWELL11	550.00	55.00	0.09	7.50E-02	0
OUTWELL12	550.00	-55.00	0.09	7.50E-02	0
OUTWELL13	550.00	65.00	0.09	7.50E-02	0
OUTWELL14	550.00	-65.00	0.09	7.50E-02	0
OUTWELL15	550.00	75.00	0.09	7.50E-02	0
OUTWELL16	550.00	-75.00	0.09	7.50E-02	0
OUTWELL17	550.00	85.00	0.09	7.50E-02	0
OUTWELL18	550.00	-85.00	0.09	7.50E-02	0
OUTWELL19	550.00	95.00	0.09	7.50E-02	0
OUTWELL20	550.00	-95.00	0.09	7.50E-02	0
OUTWELL21	550.00	105.00	0.09	7.50E-02	0
OUTWELL22	550.00	-105.00	0.09	7.50E-02	0
OUTWELL23	550.00	115.00	0.09	7.50E-02	0
OUTWELL24	550.00	-115.00	0.09	7.50E-02	0
OUTWELL25	550.00	125.00	0.09	7.50E-02	0
OUTWELL26	550.00	-125.00	0.09	7.50E-02	0
OUTWELL27	550.00	135.00	0.09	7.50E-02	0
OUTWELL28	550.00	-135.00	0.09	7.50E-02	0
OUTWELL29	550.00	145.00	0.09	7.50E-02	0
OUTWELL30	550.00	-145.00	0.09	7.50E-02	0

STREAMLINES DEPARTING FROM INJECTION WELL INWELL01

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL15	22.4 YEARS	0.0

STREAMLINES DEPARTING FROM INJECTION WELL INWELL02

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL01	21.5 YEARS	0.0

014719

... DEPARTING FROM INJECTION WELL INWELL03

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	+++none+++	50.1 YEARS	0.0

014720

STREAMLINES DEPARTING FROM INJECTION WELL [illegible]

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL02	21.5 YEARS	0.0

014721

STREAMLINES DEPARTING FROM INJECTION WELL INWELL05

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL16	22.4 YEARS	0.0

014722

EVOLUTION OF CONCENTRATION FOR PRODUCTION WELL OUTWELL01

TIME IN YEARS	CONCENTRATION IN PERCENT
21.474	0.000E-01

014723

EVOLUTION OF CONCENTRATION FOR PRODUCTION WELL OUTWELL02

TIME IN YEARS	CONCENTRATION IN PERCENT
21.474	0.000E-01

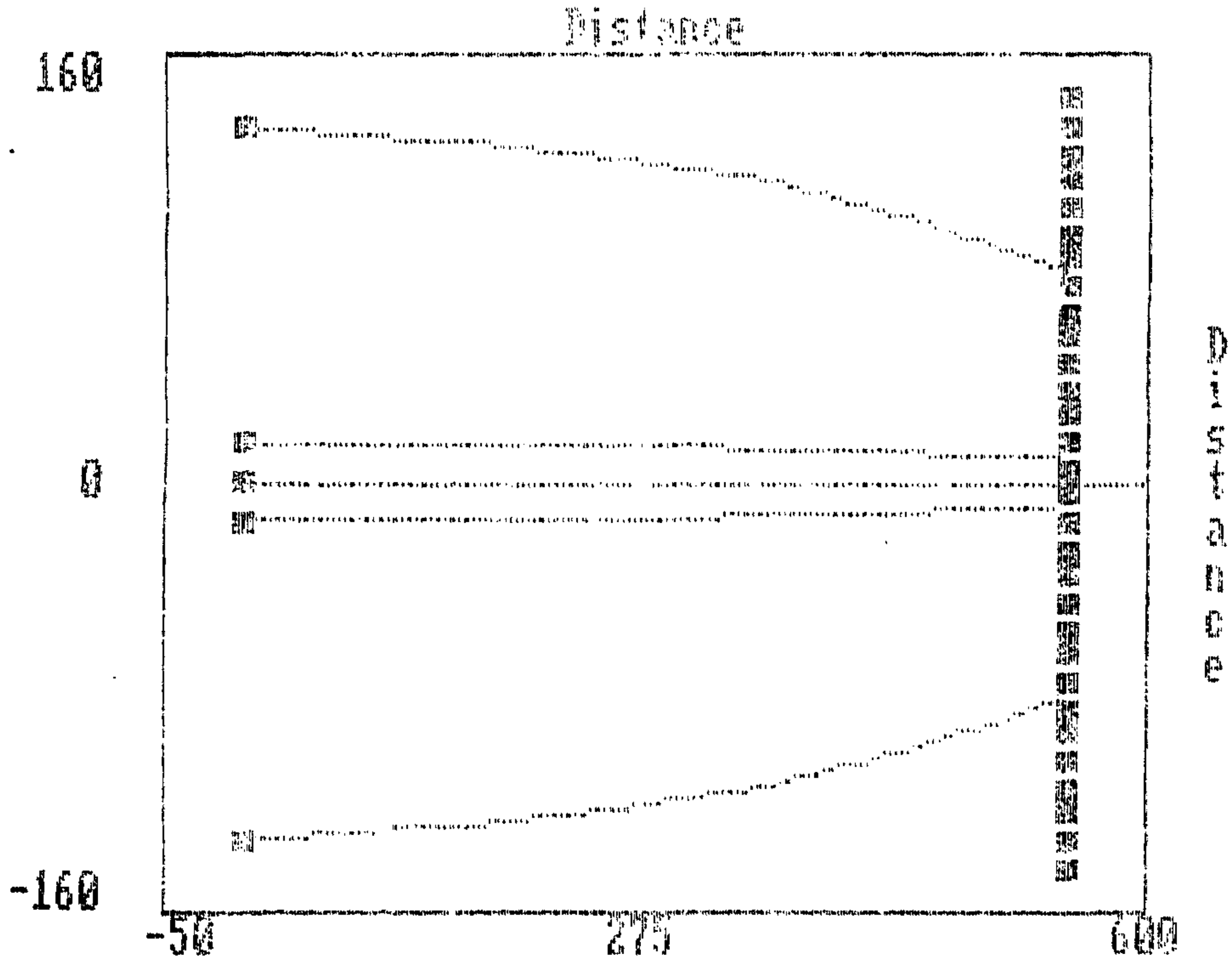
014724

EVOLUTION OF CONCENTRATION FOR PRODUCTION WELL OUTWELL15

TIME IN YEARS	CONCENTRATION IN PERCENT
22.415	0.000E-01

TIME IN YEARS	CONCENTRATION IN PERCENT
22.415	0.000E-01

014726



014727

ATTACHMENT 1C

014728

```
  5    30 C.            0.
7       0.3             0.0          0.0          0.
 INWELL01      0.      135.          .000                                  0.         1  1-1
 INWELL02     0.        15.          .000                                  0.         1  1-1
 INWELL03     0.        0.           .000                                  0.         1  1-1
 INWELL04     0.       -15.          .000                                  0.         1  1-1
 INWELL05     0.      -135.          .000                                  0.         1  1-1
OUTWELL01 550.          5.           .0909
OUTWELL02 550.         -5.           .0909
OUTWELL03 550.         15.           .0909
OUTWELL04 550.        -15.           .0909
OUTWELL05 550.         25.           .0909
OUTWELL06 550.        -25.           .0909
OUTWELL07 550.         35.           .0909
OUTWELL08 550.        -35.           .0909
OUTWELL09 550.         45.           .0909
OUTWELL10 550.        -45.           .0909
OUTWELL11 550.         55.           .0909
OUTWELL12 550.        -55.           .0909
OUTWELL13 550.         65.           .0909
OUTWELL14 550.        -65.           .0909
OUTWELL15 550.         75.           .0909
OUTWELL16 550.        -75.           .0909
OUTWELL17 550.         85.           .0909
OUTWELL18 550.        -85.           .0909
OUTWELL19 550.         95.           .0909
OUTWELL20 550.        -95.           .0909
OUTWELL21 550.        105.           .0909
OUTWELL22 550.       -105.           .0909
OUTWELL23 550.        115.           .0909
OUTWELL24 550.       -115.           .0909
OUTWELL25 550.        125.           .0909
OUTWELL26 550.       -125.           .0909
OUTWELL27 550.        135.           .0909
OUTWELL28 550.       -135.           .0909
OUTWELL29 550.        145.           .0909
OUTWELL30 550.       -145.           .0909
    1       20.
500.                               -1
-   50.      600.     - 160.      160.
```

014729

SHERIDAN - ZERO UNIFORM FLOW WITH LINE OF RECOVERY WELLS

PRACTICAL SYSTEM OF UNITS IS USED

REGIONAL FLOW, PORE VELOCITY	=	0.00 M/YR
ORIENTATION OF REGIONAL FLOW	=	0.00 DEGREES
THICKNESS OF THE AQUIFER	=	7.30 METERS
POROSITY	=	30.00 PERCENT
PERIOD STUDIED	=	500.00 YEARS
INITIAL AQUIFER CONCENTRATION	=	0.000E-01 PERCENT
DEFAULT INJECTION CONCENTRATION	=	0.000E-01 PERCENT
STREAMLINE STEP LENGTH	=	3.25 METERS
ABSORPTION CAPACITY OF ROCK	=	00.00 PERCENT

FRONTS ARE PLOTTED AT 20.000 YEARS

014730

5 INJECTION WELLS

WELL NAME	X METERS	Y METERS	FLOW-RATE M3/H	CONCENTRATION PERCENT	RADIUS METERS	INDICATOR
INWELL01	0.00	135.00	0.00	0.00E-01	7.50E-02	-1
INWELL02	0.00	15.00	0.00	0.00E-01	7.50E-02	-1
INWELL03	0.00	0.00	0.00	0.00E-01	7.50E-02	-1
INWELL04	0.00	-15.00	0.00	0.00E-01	7.50E-02	-1
INWELL05	0.00	-135.00	0.00	0.00E-01	7.50E-02	-1

30 PRODUCTION WELLS

WELL NAME	X METERS	Y METERS	FLOW-RATE M3/H	RADIUS METERS	INDICATOR
OUTWELL01	550.00	5.00	0.09	7.50E-02	0
OUTWELL02	550.00	-5.00	0.09	7.50E-02	0
OUTWELL03	550.00	15.00	0.09	7.50E-02	0
OUTWELL04	550.00	-15.00	0.09	7.50E-02	0
OUTWELL05	550.00	25.00	0.09	7.50E-02	0
OUTWELL06	550.00	-25.00	0.09	7.50E-02	0
OUTWELL07	550.00	35.00	0.09	7.50E-02	0
OUTWELL08	550.00	-35.00	0.09	7.50E-02	0
OUTWELL09	550.00	45.00	0.09	7.50E-02	0
OUTWELL10	550.00	-45.00	0.09	7.50E-02	0
OUTWELL11	550.00	55.00	0.09	7.50E-02	0
OUTWELL12	550.00	-55.00	0.09	7.50E-02	0
OUTWELL13	550.00	65.00	0.09	7.50E-02	0
OUTWELL14	550.00	-65.00	0.09	7.50E-02	0
OUTWELL15	550.00	75.00	0.09	7.50E-02	0
OUTWELL16	550.00	-75.00	0.09	7.50E-02	0
OUTWELL17	550.00	85.00	0.09	7.50E-02	0
OUTWELL18	550.00	-85.00	0.09	7.50E-02	0
OUTWELL19	550.00	95.00	0.09	7.50E-02	0
OUTWELL20	550.00	-95.00	0.09	7.50E-02	0
OUTWELL21	550.00	105.00	0.09	7.50E-02	0
OUTWELL22	550.00	-105.00	0.09	7.50E-02	0
OUTWELL23	550.00	115.00	0.09	7.50E-02	0
OUTWELL24	550.00	-115.00	0.09	7.50E-02	0
OUTWELL25	550.00	125.00	0.09	7.50E-02	0
OUTWELL26	550.00	-125.00	0.09	7.50E-02	0
OUTWELL27	550.00	135.00	0.09	7.50E-02	0
OUTWELL28	550.00	-135.00	0.09	7.50E-02	0
OUTWELL29	550.00	145.00	0.09	7.50E-02	0
OUTWELL30	550.00	-145.00	0.09	7.50E-02	0

014731

STREAMLINES DEPARTING FROM INJECTION WELL INWELL01

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL05	102.0 YEARS	0.0

STREAMLINES DEPARTING FROM INJECTION WELL INWELL02

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL01	97.7 YEARS	0.0

014733

STREAMLINES DEPARTING FROM INJECTION WELL INWELL03

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL01	100.6 YEARS	0.0

014734

STREAMLINES DEPARTING FROM INJECTION WELL INWELL04

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL02	97.7 YEARS	0.0

014735

STREAMLINES DEPARTING FROM INJECTION WELL INWELL05

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL06	102.0 YEARS	0.0

014736

EVOLUTION OF CONCENTRATION FOR PRODUCTION WELL OUTWELL01

TIME IN YEARS	CONCENTRATION IN PERCENT
97.727	0.000E-01
100.559	0.000E-01

014737

EVOLUTION OF CONCENTRATION FOR PRODUCTION WELL OUTWELL02

TIME IN YEARS	CONCENTRATION IN PERCENT
97.727	0.000E-01

EVOLUTION OF CONCENTRATION FOR PRODUCTION WELL OUTWELL05

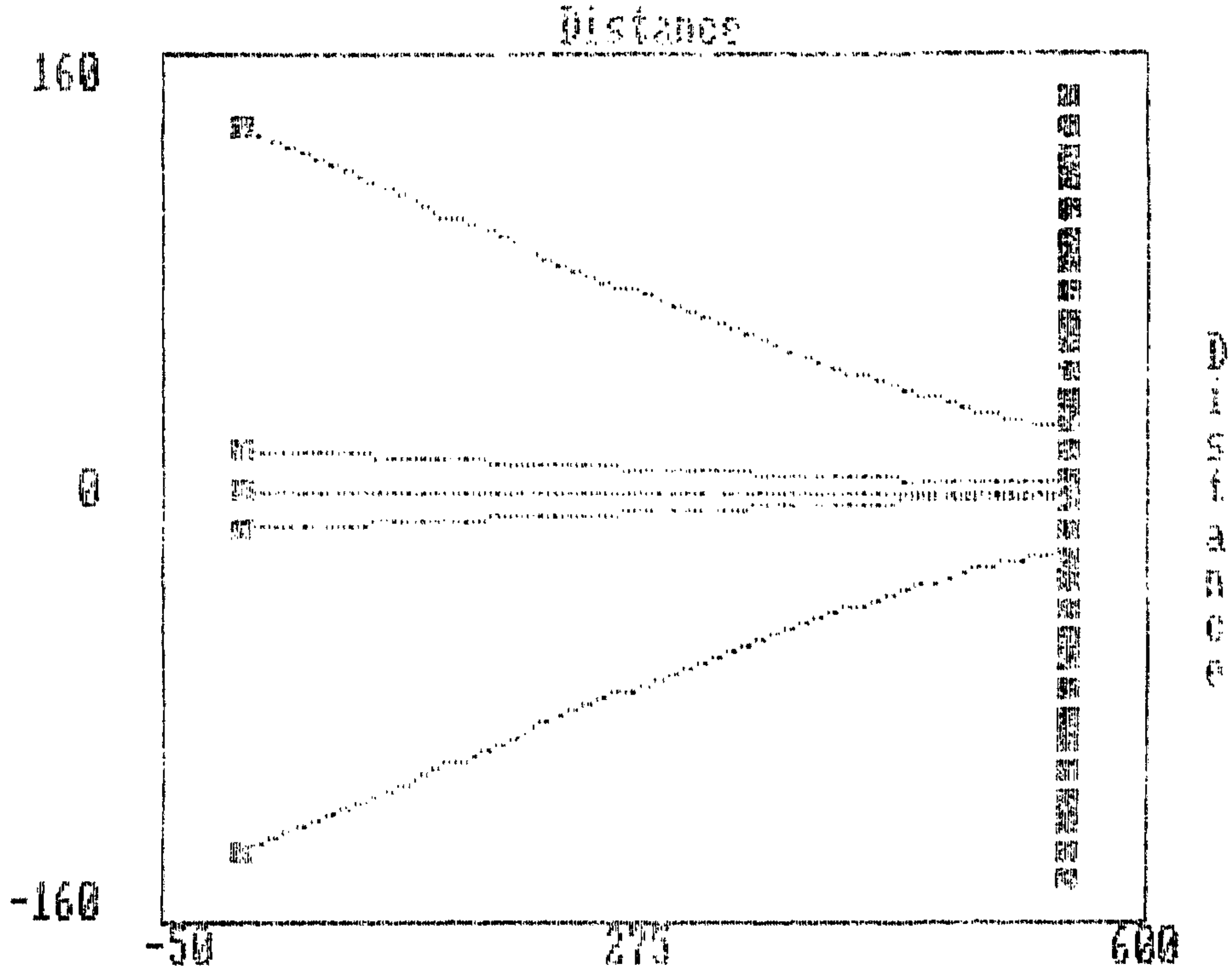
TIME IN YEARS	CONCENTRATION IN PERCENT
102.009	0.000E-01

014739

EVOLUTION OF CONCENTRATION FOR PRODUCTION WELL OUTWELL06

TIME IN YEARS	CONCENTRATION IN PERCENT
102.009	0.000E-01

014740



014741

ATTACHMENT 1D

0 1 4 7 4 2

```
    5     6 0.          0.
 3        0.3       00.0       0.0       0.
 INWELL01    0.     135.        .000                                    0.        1 1-1
 NWELL02    0.      15.        .000                                    0.        1 1-1
 NWELL03    0.       0.        .000                                    0.        1 1-1
 INWELL04    0.     -15.        .000                                    0.        1 1-1
 NWELL05    0.    -135.        .000                                    0.        1 1-1
UTWELL25 550.       0.          .463
OUTWELL26 560.       0.          .463
 TWELL27 550.      145.         .463
OUTWELL28 560.      145.         .463
OUTWELL29 550.     -145.         .463
 TWELL30 560.     -145.         .463
   1      20.
00.                          -1
     -50.       600.      -160.       160.
```

014743

SHERIDAN - ZERO UNIFORM FLOW WITH PARTIAL SLURRY WALL AND EXTRACTION WELLS

PRACTICAL SYSTEM OF UNITS IS USED

```
REGIONAL FLOW, PORE VELOCITY         =       0.00 M/YR
ORIENTATION OF REGIONAL FLOW         =       0.00 DEGREES
THICKNESS OF THE AQUIFER             =       7.30 METERS
POROSITY                             =      30.00 PERCENT
PERIOD STUDIED                       =     500.00 YEARS
INITIAL AQUIFER CONCENTRATION        =  0.000E-01 PERCENT
DEFAULT INJECTION CONCENTRATION      =  0.000E-01 PERCENT
STREAMLINE STEP LENGTH               =       3.25 METERS
ADSORPTION CAPACITY OF ROCK          =      00.00 PERCENT
```

1 FRONTS ARE PLOTTED AT 20.000 YEARS

014744

5 INJECTION WELLS

WELL NAME	X METERS	Y METERS	FLOW-RATE M3/H	CONCENTRATION PERCENT	RADIUS METERS	INDICATOR
INWELL01	0.00	135.00	0.00	0.00E-01	7.50E-02	-1
INWELL02	0.00	15.00	0.00	0.00E-01	7.50E-02	-1
INWELL03	0.00	0.00	0.00	0.00E-01	7.50E-02	-1
INWELL04	0.00	-15.00	0.00	0.00E-01	7.50E-02	-1
INWELL05	0.00	-135.00	0.00	0.00E-01	7.50E-02	-1

6 PRODUCTION WELLS

WELL NAME	X METERS	Y METERS	FLOW-RATE M3/H	RADIUS METERS	INDICATOR
OUTWELL25	550.00	0.00	0.46	7.50E-02	0
OUTWELL26	560.00	0.00	0.46	7.50E-02	0
OUTWELL27	550.00	145.00	0.46	7.50E-02	0
OUTWELL28	560.00	145.00	0.46	7.50E-02	0
OUTWELL29	550.00	-145.00	0.46	7.50E-02	0
OUTWELL30	560.00	-145.00	0.46	7.50E-02	0

TREAMLINES DEPARTING FROM INJECTION WELL INWELL01

NUMBER OF TREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL25	107.2 YEARS	0.0

014746

TREAMLINES DEPARTING FROM INJECTION WELL INWELL02

NUMBER OF TREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL25	102.0 YEARS	0.0

014747

TREAMLINES DEPARTING FROM INJECTION WELL INWELL03

NUMBER OF TREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL25	101.9 YEARS	0.0

STREAMLINES DEPARTING FROM INJECTION WELL INWELL04

NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL25	102.0 YEARS	0.0

014749

STREAMLINES DEPARTING FROM INJECTION WELL INWELL05

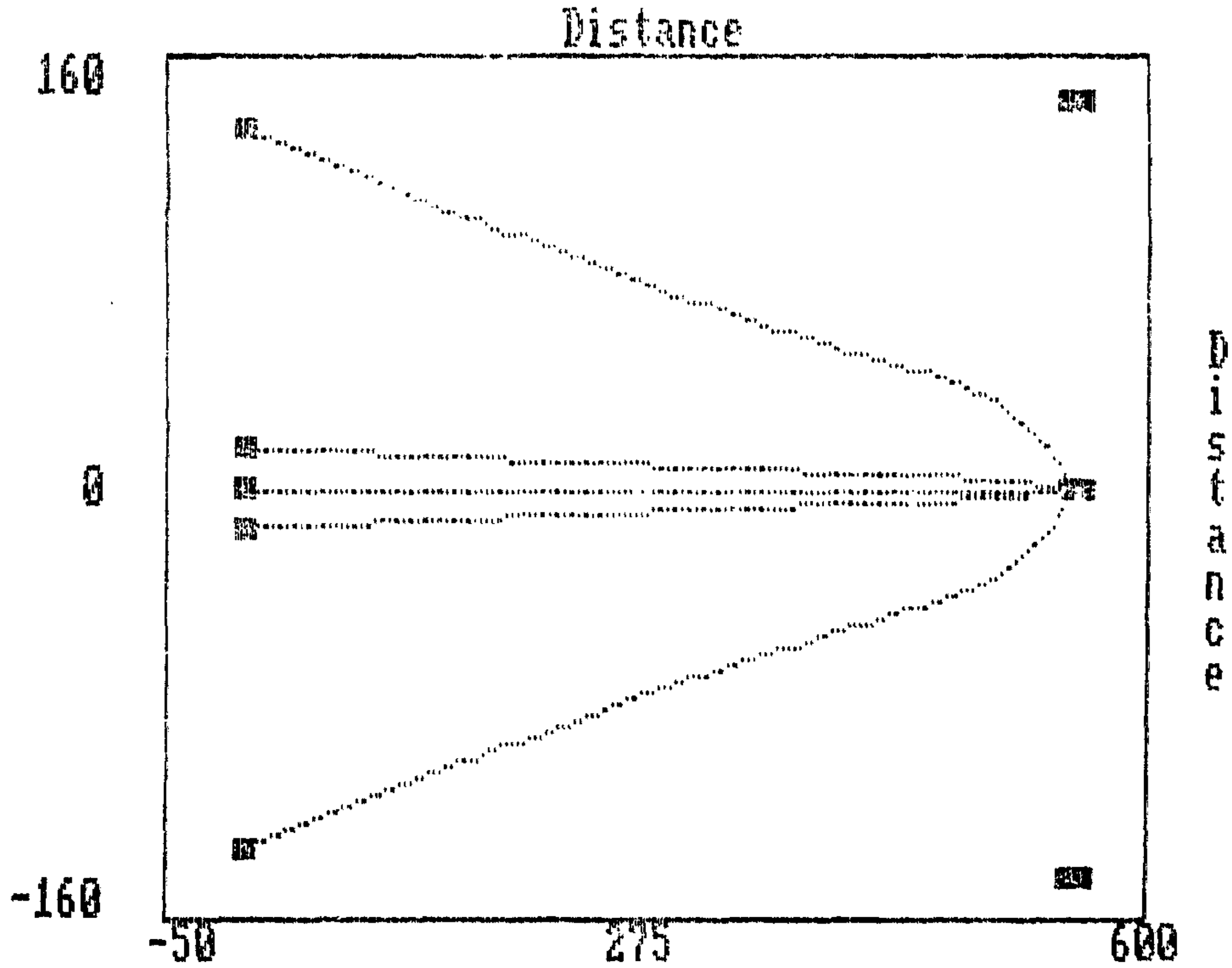
NUMBER OF STREAMLINE	WELL REACHED	TIME OF ARRIVAL	ANGLE BETA IN DEGREES
1	OUTWELL25	107.2 YEARS	0.0

014750

VOLUTION OF CONCENTRATION FOR PRODUCTION WELL OUTWELL25

TIME IN YEARS	CONCENTRATION IN PERCENT
102.020	0.000E-01
107.219	0.000E-01

014751



014752

Attachment 2
Sorption of Benzene and Trichloroethylene

The effective sorption of contaminants within an aquifer can be represented in a number of ways. The simplest approach, and one which is commonly used in mathematical modeling of contaminant transport, is to develop a retardation coefficient which expresses the rate of transport of a specific contaminant as a proportion of the pore velocity of groundwater flow. The retardation coefficient (reciprocal of the relative velocity) is given by (Walton, Practical Aspects of Ground Water Modeling, National Water Well Association, 1985)

$$R_d = 1 + (\text{rho}\ K_d/n),$$

where rho is the bulk mass density of aquifer skeleton, K_d is the distribution coefficient for the contaminant in question, and n is the porosity of the porous medium.

The distrubution coefficient for sorption of organic chemicals onto the organic carbon of the porous medium can be given by (Walton, op. cit.)

$$K_d = 0.63\ f_{oc}\ K_{ow}\ (\text{ml/g}),$$

where f_{oc} is the organic carbon content of the medium and K_{ow} is the octanol-water partition coefficent.

For benzene and trichloroethylene (TCE), values of K_{ow} are readily available in the literature (Walton op. cit.):

benzene: $\log K_{ow} = 2.13$

TCE: $\log K_{ow} = 2.29$

For the Sheridan site, organic carbon content of the soils has not been measured, although estimates of cation exchange capacity (CEC) are available. For a well-humified soil, a very general empirical relationship between organic carbon content and CEC is available in the literature. However, the shallow water-table aquifer at the Sheridan site probably does not contain large amounts of humic material. In general, organic carbon content of natural soils ranges from a fraction of a percent to a few percent. For purposes of the present estimates, let us assume that the organic carbon content of the shallow aquifer materials at the Sheridan site may be in the range of 0.0005 to 0.05. The corresponding ranges in values of K_d are then:

Benzene: 0.0425 to 4.25 ml/g

TCE: 0.0614 to 6.14 ml/g

For a porosity of 0.3 and a bulk density of 1.75 g/ml, the corresponding ranges of retardation coefficients are:

Benzene: 1.25 to 25.8

TCE: 1.36 to 36.8

Thus, in general, we can expect these contaminants to travel about one to 30 times more slowly than groundwater.

APPENDIX B

Estimation of Drawdowns Along a Hypothetical Line of Recovery Wells

APPENDIX B

ESTIMATION OF DRAWDOWNS
ALONG A HYPOTHETICAL LINE OF RECOVERY WELLS

GROUND WATER FEASIBILITY STUDY
SHERIDAN DISPOSAL SERVICES
HEMPSTEAD, TEXAS

INTRODUCTION

One remediation option proposed for the Sheridan Disposal Services site is a system of recovery wells at a 20-foot spacing, installed between the main pond and the Brazos River. A series of analytical calculations has been performed to demonstrate the effect of well spacing on drawdowns along the line of wells.

An analytical solution, based on the Theis equation, was developed to estimate drawdown near a line of adjacent recovery wells.

For ease of calculation, the solution assumes the aquifer thickness does not vary with declining water levels, although the aquifer observed at the Sheridan site is unconfined.

The transmissivity of the actual aquifer will decrease as its saturated thickness decreases, and as a result a greater hydraulic gradient and greater drawdowns (than in the assumed confined aquifer) will be required to maintain a given discharge rate, or a smaller discharge rate will be possible for a given drawdown. As a result, this solution underestimates drawdowns for a given well discharge rate, or conversely overestimates possible discharge rates for a given limiting drawdown. The solution is presented in the form of a spreadsheet. The basic equations and assumptions used are shown in Attachment 1, and results are given in Attachments 2(A) and 2(B).

SELECTION OF PARAMETERS

For this calculation a transmissivity of 4,000 gpd/ft was used, based on the pumping test data developed in the Source Control Remedial Investigation (July 1987) for the unconfined aquifer. A specific yield or storage coefficient of 0.001 was assumed. A discharge rate (Q_{well}) of 0.24 gpm per recovery well was used, as this is sufficient to capture the ground water flow between two adjacent recovery wells. The ground water flow is estimated at 17 gpd/foot, measured perpendicular to the direction of ground water flow.

L633

The line of wells is assumed to lie 100 feet away from the riverbank. The river may or may not act as a recharge boundary, depending on relative elevations of the river stage and the ground water table as affected by pumping. If the river level is higher than the ground water table, so that the river forms a recharge boundary, the boundary can be simulated by a series of imaginary recharge wells, each one opposite the corresponding recovery well, (see Attachment 1). In each recharge well, a negative discharge rate (Q_{image}) equal and opposite in sign to Q_{real} is used. If the river is lower than the ground water table, a value of Q_{image} closer to zero is more appropriate. All the wells are assumed to have been pumping for 30 years, the approximate amount of time calculated for a particle of water to travel northward from the southern end of the main pond to the pumping wells.

RESULTS

Attachments 2(A) and 2(B) show solutions for the drawdowns in the line of recovery wells at the site, with and without recharge from the Brazos River. The well number (i) refers to the position of the pair of wells (recovery and imaginary recharge well, if used) next to point "0", at which the drawdown observation is made. The dimensionless factor "u" is calculated based on the equation

$$u \quad = \quad r^2 * S \; / \; (4 * T * t)$$

and the variable W(u) is the well function of u. The drawdown increment is the amount of drawdown due to each well in the line of wells, and is equal to delta-s, where

$$\text{delta-s} \quad = \quad Q * W(u) \; / \; (4 * Pi * T).$$

In the next column, these increments have been summed to show the drawdown at the end of a line of several wells, taking the corresponding image wells into account if needed, but ignoring the effects of the remaining wells. The drawdown at some observation point within the 50 recovery wells can then be calculated by considering separately the two lines of wells to either side of the observation point, and adding the drawdowns calculated for the end of the two lines. The sum is shown in the last column.

Attachment 2A shows the calculated drawdown assuming that the river is not acting as a recharge boundary hence Q_{image} equals zero. The table shows that near the center of the line of 50 pumping wells, the drawdown at the edge of the borehole would be about 4.4 feet. Taking into account the change in saturated thickness (see Equation 1 in Attachment 2A), the actual drawdown at this point would be approximately 5 feet. If it is assumed that the recovery well is 20% efficient (a reasonable assumption), then the actual drawdown

L633

in the well itself would be about 25 feet. The available saturated thickness of the aquifer generally varies between 20 and 25 feet at the site. Therefore, the assumption of 20 foot well spacings and a discharge rate of approximately 0.24 gpm would be the well design needed to capture ground water flowing into the Brazos River. If a higher pumping rate was chosen, the well spacing could be decreased, but the recovery wells would most likely be pumped dry prior to maintaining a steady discharge and cone of depression.

Summary

Calculations of the effectiveness of a line of recovery wells adjacent to the Brazos River have been completed. If no recharge from the Brazos River is assumed, the calculations show that drawdown from intersecting cones of depression allow capture of contaminated ground water from the site. This is accomplished at a well spacing of 20 feet and an average discharge of about 0.24 gpm, and effectively ceases the migration of contaminated ground water to the river. If recharge equal to the recovery well discharge is assumed, there is still ground water capture into the recovery wells. At higher flow (recharge) from the Brazos River, the water levels in the aquifer would rise, causing a change in flow direction to the south, thereby also precluding constituent migration into the river.

In contrast, when the river is assumed to act as a recharge boundary, the calculated drawdown is greatly reduced and would be only about 0.5 feet maximum (Attachment 2B). This calculation assumes that the input from the river equals 0.24 gpm per well.

L633

ATTACHMENT 1

014759

L833

Estimate of Maximum
Possible Discharge in Dewatering Wells
Ground Water Feasibility Study
Sheridan Disposal Services Site

Schematic Drawing Only (not drawn to scale)

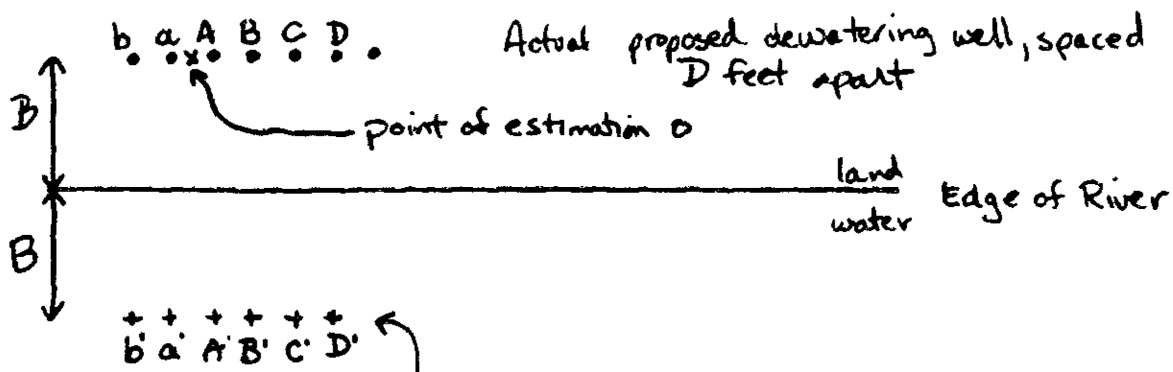


Image "Recharge" Wells to simulate constant-head boundary at river's edge.

014760

Assumptions:

(1) Effect of well a, to left of Point of Estimation, will be the same as that of A, to the right.

(2) Aquifer behaves like a confined aquifer, that is the saturated thickness is assumed not to change, so that T doesn't change, and the equation is valid. If the aquifer is unconfined, drawdowns will actually be greater or available discharge will be less.

(3) There is a constant-head boundary at the river's edge.

(4) Aquifer parameters:

$T = 4 \times 10^3$ gpd/ft (based on SCRI)
Max. available drawdown = 24' = Aquifer Thickness

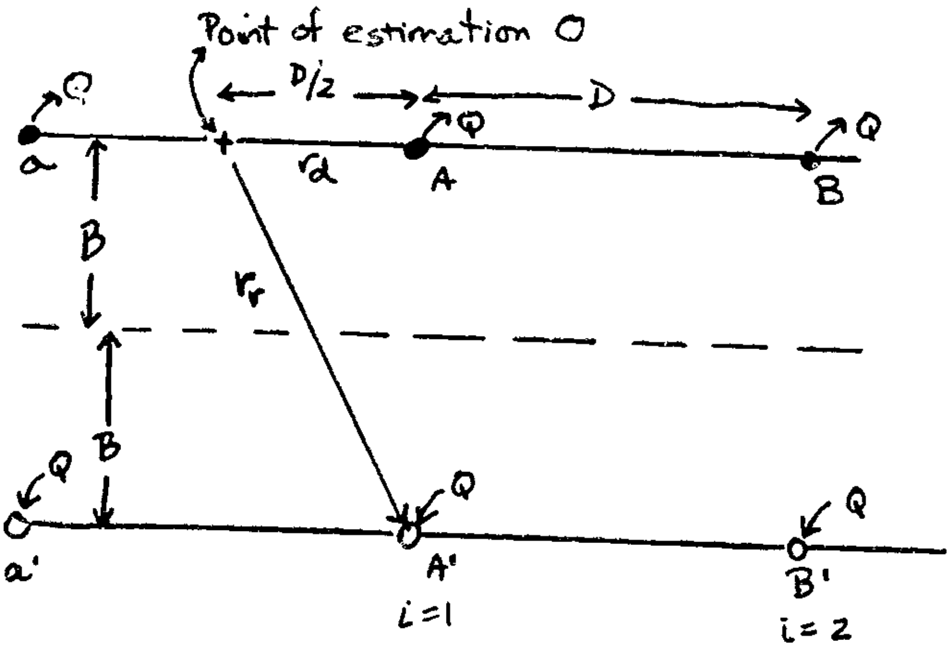
(5) Wells are spaced D feet apart, and located in a line B feet away from the river.

(6) Net recharge other than from the river can be ignored for the [illegible] period in question.

[illegible]downs from adjacent wells can be superimposed.

L766

Schematic Drawing Only (not to scale)



014761

for i^{th} pair of wells

$r_d = D(i-\frac{1}{2})$

$r_r = [(2B)^2 + r_d^2]^{\frac{1}{2}}$

We want drawdown increment from i^{th} pair of wells

$$s = \frac{Q\,W\,(u)}{4\,\pi\,T}$$

where $(u) = \dfrac{r^2 S}{4Tt}$

[r] = ft
[S] = 1
[T] = gpd/ft
[t] = days

L765

ATTACHMENT 2A

Drawdown Assuming No Recharge from Brazos River

014762

L533

ATTACHMENT 2A

ESTIMATE OF DRAWDOWN FROM PUMPING IN RECOVERY WELLS
ASSUMING CONSTANT SATURATED THICKNESS
SHERIDAN DISPOSAL SERVICES, HEMPSTEAD, TEXAS

PARAMETERS USED

Trans.	: 4000	(gpd/ft)
S	: 1.00E-03	
Qreal	: 0.24	(gpm)
Qimage	: 0.00	(gpm)
Time	:365*30	(days)

GEOMETRY

WELL SPACING D (Ft)	20
DIST. TO RIVER B (Ft)	100
DISCHARGE RATE (gpm) TO CAPTURE 17 gpd/ft	0.24

DRAWDOWN ESTIMATED FOR POINT BETWEEN TWO WELLS

RECOVERY OR IMAGE WELL NO. (I)	DISCHARGE (gpm)	DIST. FROM O TO WELL (Ft)	u ( )	W(u) ( )	DRAWDOWN INCREMENT (Ft)	CUMULATIVE DRAWDOWN AT END OF LINE OF "I" WELLS (Ft)	DRAWDOWN 10' PAST WELL "I" (Ft)
1	0.24	10.0	4.27E-09	1.87E+01	0.126	0.126	3.942
1	0.00	200.2	1.71E-06	1.27E+01	0.000	0.126	3.942
2	0.24	30.0	3.84E-08	1.65E+01	0.112	0.238	3.989
2	0.00	202.2	1.75E-06	1.27E+01	0.000	0.238	3.989
3	0.24	50.0	1.07E-07	1.55E+01	0.105	0.343	4.028
3	0.00	206.2	1.81E-06	1.26E+01	0.000	0.343	4.028
4	0.24	70.0	2.09E-07	1.48E+01	0.100	0.443	4.063
4	0.00	211.9	1.92E-06	1.26E+01	0.000	0.443	4.063
5	0.24	90.0	3.46E-07	1.43E+01	0.097	0.540	4.095
5	0.00	219.3	2.05E-06	1.25E+01	0.000	0.540	4.095
6	0.24	110.0	5.17E-07	1.39E+01	0.094	0.634	4.123
6	0.00	228.3	2.22E-06	1.24E+01	0.000	0.634	4.123
7	0.24	130.0	7.22E-07	1.36E+01	0.092	0.725	4.149
7	0.00	238.5	2.43E-06	1.24E+01	0.000	0.725	4.149
8	0.24	150.0	9.61E-07	1.33E+01	0.090	0.815	4.172
8	0.00	250.0	2.67E-06	1.23E+01	0.000	0.815	4.172
9	0.24	170.0	1.23E-06	1.30E+01	0.088	0.903	4.194
9	0.00	262.5	2.94E-06	1.22E+01	0.000	0.903	4.194
10	0.24	190.0	1.54E-06	1.28E+01	0.087	0.990	4.213
10	0.00	275.9	3.25E-06	1.21E+01	0.000	0.990	4.213
11	0.24	210.0	1.88E-06	1.26E+01	0.085	1.075	4.231
11	0.00	290.0	3.59E-06	1.20E+01	0.000	1.075	4.231
12	0.24	230.0	2.26E-06	1.24E+01	0.084	1.159	4.247
12	0.00	304.8	3.97E-06	1.19E+01	0.000	1.159	4.247
13	0.24	250.0	2.67E-06	1.23E+01	0.083	1.242	4.262
13	0.00	320.2	4.38E-06	1.18E+01	0.000	1.242	4.262
14	0.24	270.0	3.11E-06	1.21E+01	0.082	1.324	4.276
14	0.00	336.0	4.82E-06	1.17E+01	0.000	1.324	4.276
15	0.24	290.0	3.59E-06	1.20E+01	0.081	1.405	4.288
15	0.00	352.3	5.30E-06	1.16E+01	0.000	1.405	4.288
16	0.24	310.0	4.10E-06	1.18E+01	0.080	1.485	4.299
16	0.00	368.9	5.81E-06	1.15E+01	0.000	1.485	4.299
17	0.24	330.0	4.65E-06	1.17E+01	0.079	1.564	4.308
17	0.00	385.9	6.36E-06	1.14E+01	0.000	1.564	4.308
18	0.24	350.0	5.23E-06	1.16E+01	0.078	1.642	4.317
18	0.00	403.1	6.94E-06	1.13E+01	0.000	1.642	4.317
19	0.24	370.0	5.84E-06	1.15E+01	0.078	1.720	4.324
19	0.00	420.6	7.55E-06	1.12E+01	0.000	1.720	4.324
20	0.24	390.0	6.49E-06	1.14E+01	0.077	1.797	4.330
20	0.00	438.3	8.20E-06	1.11E+01	0.000	1.797	4.330
21	0.24	410.0	7.18E-06	1.13E+01	0.076	1.873	4.335
21	0.00	456.2	8.88E-06	1.11E+01	0.000	1.873	4.335
22	0.24	430.0	7.89E-06	1.12E+01	0.076	1.949	4.339
22	0.00	474.2	9.60E-06	1.10E+01	0.000	1.949	4.339
23	0.24	450.0	8.65E-06	1.11E+01	0.075	2.024	4.341
23	0.00	492.4	1.04E-05	1.09E+01	0.000	2.024	4.341
24	0.24	470.0	9.43E-06	1.10E+01	0.074	2.098	4.343
24	0.00	510.8	1.11E-05	1.08E+01	0.000	2.098	4.343
25	0.24	490.0	1.03E-05	1.09E+01	0.074	2.172	4.344
25	0.00	529.2	1.20E-05	1.08E+01	0.000	2.172	4.344

014763

ATTACHMENT 2A (continued)

ESTIMATE OF DRAWDOWN FROM PUMPING IN RECOVERY WELLS
ASSUMING CONSTANT SATURATED THICKNESS
SHERIDAN DISPOSAL SERVICES, HEMPSTEAD, TEXAS

26	0.24	510.0	1.11E-05	1.08E+01	0.073	2.245	4.343
26	0.00	547.8	1.28E-05	1.07E+01	0.000	2.245	4.343
27	0.24	530.0	1.20E-05	1.08E+01	0.073	2.318	4.341
27	0.00	566.5	1.37E-05	1.06E+01	0.000	2.318	4.341
28	0.24	550.0	1.29E-05	1.07E+01	0.072	2.390	4.339
28	0.00	585.2	1.46E-05	1.06E+01	0.000	2.390	4.339
29	0.24	570.0	1.39E-05	1.06E+01	0.072	2.462	4.335
29	0.00	604.1	1.56E-05	1.05E+01	0.000	2.462	4.335
30	0.24	590.0	1.49E-05	1.05E+01	0.071	2.533	4.330
30	0.00	623.0	1.66E-05	1.04E+01	0.000	2.533	4.330
31	0.24	610.0	1.59E-05	1.05E+01	0.071	2.604	4.324
31	0.00	642.0	1.76E-05	1.04E+01	0.000	2.604	4.324
32	0.24	630.0	1.69E-05	1.04E+01	0.070	2.674	4.317
32	0.00	661.0	1.87E-05	1.03E+01	0.000	2.674	4.317
33	0.24	650.0	1.80E-05	1.03E+01	0.070	2.744	4.308
33	0.00	680.1	1.97E-05	1.03E+01	0.000	2.744	4.308
34	0.24	670.0	1.92E-05	1.03E+01	0.070	2.814	4.299
34	0.00	699.2	2.09E-05	1.02E+01	0.000	2.814	4.299
35	0.24	690.0	2.03E-05	1.02E+01	0.069	2.883	4.288
35	0.00	718.4	2.20E-05	1.01E+01	0.000	2.883	4.288
36	0.24	710.0	2.15E-05	1.02E+01	0.069	2.952	4.276
36	0.00	737.6	2.32E-05	1.01E+01	0.000	2.952	4.276
37	0.24	730.0	2.28E-05	1.01E+01	0.068	3.020	4.262
37	0.00	756.9	2.45E-05	1.00E+01	0.000	3.020	4.262
38	0.24	750.0	2.40E-05	1.01E+01	0.068	3.088	4.247
38	0.00	776.2	2.57E-05	9.99E+00	0.000	3.088	4.247
39	0.24	770.0	2.53E-05	1.00E+01	0.068	3.156	4.231
39	0.00	795.6	2.70E-05	9.94E+00	0.000	3.156	4.231
40	0.24	790.0	2.66E-05	9.96E+00	0.067	3.223	4.213
40	0.00	814.9	2.84E-05	9.89E+00	0.000	3.223	4.213
41	0.24	810.0	2.80E-05	9.91E+00	0.067	3.290	4.194
41	0.00	834.3	2.97E-05	9.85E+00	0.000	3.290	4.194
42	0.24	830.0	2.94E-05	9.86E+00	0.067	3.357	4.172
42	0.00	853.8	3.11E-05	9.80E+00	0.000	3.357	4.172
43	0.24	850.0	3.08E-05	9.81E+00	0.066	3.423	4.149
43	0.00	873.2	3.26E-05	9.76E+00	0.000	3.423	4.149
44	0.24	870.0	3.23E-05	9.76E+00	0.066	3.489	4.123
44	0.00	892.7	3.40E-05	9.71E+00	0.000	3.489	4.123
45	0.24	890.0	3.38E-05	9.72E+00	0.066	3.555	4.095
45	0.00	912.2	3.55E-05	9.67E+00	0.000	3.555	4.095
46	0.24	910.0	3.54E-05	9.67E+00	0.065	3.621	4.063
46	0.00	931.7	3.71E-05	9.63E+00	0.000	3.621	4.063
47	0.24	930.0	3.69E-05	9.63E+00	0.065	3.686	4.028
47	0.00	951.3	3.86E-05	9.58E+00	0.000	3.686	4.028
48	0.24	950.0	3.85E-05	9.59E+00	0.065	3.750	3.989
48	0.00	970.8	4.02E-05	9.54E+00	0.000	3.750	3.989
49	0.24	970.0	4.02E-05	9.55E+00	0.065	3.815	3.942
49	0.00	990.4	4.19E-05	9.50E+00	0.000	3.815	3.942
50	0.24	990.0	4.18E-05	9.50E+00	0.064	3.879	3.879
50	0.00	1010.0	4.36E-05	9.46E+00	0.000	3.879	3.879

DRAWDOWN ESTIMATED 0.25 Ft. FROM CENTER OF PUMPING WELL

RECOVERY OR IMAGE WELL NO. (i)	DISCHARGE (gpm)	DIST. FROM O TO WELL (Ft)	u ( )	W(u) ( )	DRAWDOWN INCREMENT (Ft)	CUMULATIVE DRAWDOWN AT END OF LINE OF "I" WELLS (Ft)
0	0.24	0.25	2.67E-12	2.61E+01	0.176	4.446

014764

L757 ATTACHMENT 2A

EQUATION 1:
Correction for Drawdown

S	B	S'=S-(S^2/2/B)
0	24	0.00
1	24	0.98
2	24	1.92
3	24	2.81
4	24	3.67
5	24	4.48
6	24	5.25
7	24	5.98
8	24	6.67
9	24	7.31
10	24	7.92
11	24	8.48
12	24	9.00
13	24	9.48
14	24	9.92
15	24	10.31
16	24	10.67
17	24	10.98
18	24	11.25
19	24	11.48
20	24	11.67
21	24	11.81
22	24	11.92
23	24	11.98
24	24	12.00

Where S = drawdown
S' = corrected drawdown
B = saturated thickness of aquifer

014765

ATTACHMENT 2B

Drawdown Assuming Recharge from the Brazos River

014766

L533

ATTACHMENT 2B

ESTIMATE OF DRAWDOWN FROM PUMPING IN RECOVERY WELLS ASSUMING CONSTANT SATURATED THICKNESS SHERIDAN DISPOSAL SERVICES, HEMPSTEAD, TEXAS

PARAMETERS USED

Trans.	: 4000	(gpd/ft)
S	: 1.00E-03	
Qreal	: 0.24	(gpm)
Qimage	: -0.24	(gpm)
Time	: 365*30	(days)

GEOMETRY

WELL SPACING D (Ft)	20
DIST. TO RIVER B (Ft)	100
DISCHARGE RATE (gpm) TO CAPTURE 17 gpd/ft	0.24

DRAWDOWN ESTIMATED FOR POINT BETWEEN TWO WELLS

RECOVERY OR IMAGE WELL NO. (I)	DISCHARGE (gpm)	DIST. FROM O TO WELL (Ft)	u ( )	W(u) ( )	DRAWDOWN INCREMENT (Ft)	CUMULATIVE DRAWDOWN AT END OF LINE OF "I" WELLS (Ft)	DRAWDOWN 10' PAST WELL "I" (Ft.)
1	-0.24	200.2	1.71E-06	1.27E+01	-0.086	0.041	0.299
2	0.24	30.0	3.84E-08	1.65E+01	0.112	0.152	0.411
2	-0.24	202.2	1.75E-06	1.27E+01	-0.086	0.066	0.325
3	0.24	50.0	1.07E-07	1.55E+01	0.105	0.171	0.429
3	-0.24	206.2	1.81E-06	1.26E+01	-0.086	0.086	0.344
4	0.24	70.0	2.09E-07	1.48E+01	0.100	0.186	0.444
4	-0.24	211.9	1.92E-06	1.26E+01	-0.085	0.101	0.359
5	0.24	90.0	3.46E-07	1.43E+01	0.097	0.197	0.456
5	-0.24	219.3	2.05E-06	1.25E+01	-0.085	0.113	0.371
6	0.24	110.0	5.17E-07	1.39E+01	0.094	0.207	0.465
6	-0.24	228.3	2.22E-06	1.24E+01	-0.084	0.122	0.381
7	0.24	130.0	7.22E-07	1.36E+01	0.092	0.214	0.472
7	-0.24	238.5	2.43E-06	1.24E+01	-0.084	0.131	0.389
8	0.24	150.0	9.61E-07	1.33E+01	0.090	0.220	0.479
8	-0.24	250.0	2.67E-06	1.23E+01	-0.083	0.138	0.396
9	0.24	170.0	1.23E-06	1.30E+01	0.088	0.226	0.484
9	-0.24	262.5	2.94E-06	1.22E+01	-0.082	0.143	0.402
10	0.24	190.0	1.54E-06	1.28E+01	0.087	0.230	0.488
10	-0.24	275.9	3.25E-06	1.21E+01	-0.082	0.148	0.406
11	0.24	210.0	1.88E-06	1.26E+01	0.085	0.234	0.492
11	-0.24	290.0	3.59E-06	1.20E+01	-0.081	0.153	0.411
12	0.24	230.0	2.26E-06	1.24E+01	0.084	0.237	0.495
12	-0.24	304.8	3.97E-06	1.19E+01	-0.080	0.157	0.414
13	0.24	250.0	2.67E-06	1.23E+01	0.083	0.240	0.497
13	-0.24	320.2	4.38E-06	1.18E+01	-0.080	0.160	0.418
14	0.24	270.0	3.11E-06	1.21E+01	0.082	0.242	0.499
14	-0.24	336.0	4.82E-06	1.17E+01	-0.079	0.163	0.420
15	0.24	290.0	3.59E-06	1.20E+01	0.081	0.244	0.501
15	-0.24	352.3	5.30E-06	1.16E+01	-0.078	0.166	0.423
16	0.24	310.0	4 10E-06	1.18E+01	0.080	0.246	0.503
16	-0.24	368.9	5.81E-06	1.15E+01	-0.078	0.168	0.425
17	0.24	330.0	4.65E-06	1.17E+01	0.079	0.247	0.504
17	-0 24	385.9	6.36E-06	1.14E+01	-0.077	0.170	0.427
18	0.24	350.0	5.23E-06	1.16E+01	0.078	0.248	0.505
18	-0.24	403.1	6.94E-06	1.13E+01	-0.076	0.172	0.429
19	0.24	370.0	5.84E-06	1.15E+01	0.078	0.250	0.506
19	-0.24	420.6	7.55E-06	1.12E+01	-0.076	0.174	0.430
20	0.24	390.0	6.49E-06	1.14E+01	0.077	0.251	0.507
20	-0.24	438.3	8.20E-06	1.11E+01	-0.075	0.175	0.432
21	0.24	410.0	7.18E-06	1.13E+01	0.076	0.251	0.507
21	-0.24	456.2	8.88E-06	1.11E+01	-0.075	0.177	0.433
22	0.24	430.0	7.89E-06	1.12E+01	0.076	0.252	0.508
22	-0.24	474.2	9.60E-06	1.10E+01	-0.074	0.178	0.434
23	0.24	450.0	8.65E-06	1.11E+01	0.075	0.253	C.508
23	-0.24	492.4	1.04E-05	1.09E+01	-0.074	0.179	0.434
24	0.24	470.0	9.43E-06	1.10E+01	0.074	0.254	0.508
24	-0.24	510.8	1.11E-05	1.08E+01	-0.073	0.180	0.435
25	0.24	490.0	1.03E-05	1.09E+01	0.074	0.254	0.508
25	-0.24	529.2	1.20E-05	1.08E+01	-0.073	0.181	0.436

014767

ATTACHMENT 2B (continued)

ESTIMATE OF DRAWDOWN FROM PUMPING IN RECOVERY WELLS
ASSUMING CONSTANT SATURATED THICKNESS
SHERIDAN DISPOSAL SERVICES, HEMPSTEAD, TEXAS

26	0.24	510.0	1.11E-05	1.08E+01	0.073	0.255	0.508
26	-0.24	547.8	1.28E-05	1.07E+01	-0.072	0.182	0.436
27	0.24	530.0	1.20E-05	1.08E+01	0.073	0.255	0.508
27	-0.24	566.5	1.37E-05	1.06E+01	-0.072	0.183	0.436
28	0.24	550.0	1.29E-05	1.07E+01	0.072	0.256	0.508
28	-0.24	585.2	1.46E-05	1.06E+01	-0.071	0.184	0.436
29	0.24	570.0	1.39E-05	1.06E+01	0.072	0.256	0.507
29	-0.24	604.1	1.56E-05	1.05E+01	-0.071	0.185	0.436
30	0.24	590.0	1.49E-05	1.05E+01	0.071	0.256	0.507
30	-0.24	623.0	1.66E-05	1.04E+01	-0.071	0.186	0.436
31	0.24	610.0	1.59E-05	1.05E+01	0.071	0.257	0.506
31	-0.24	642.0	1.76E-05	1.04E+01	-0.070	0.186	0.436
32	0.24	630.0	1.69E-05	1.04E+01	0.070	0.257	0.505
32	-0.24	661.0	1.87E-05	1.03E+01	-0.070	0.187	0.435
33	0.24	650.0	1.80E-05	1.03E+01	0.070	0.257	0.504
33	-0.24	680.1	1.97E-05	1.03E+01	-0.069	0.188	0.435
34	0.24	670.0	1.92E-05	1.03E+01	0.070	0.257	0.503
34	-0.24	699.2	2.09E-05	1.02E+01	-0.069	0.188	0.434
35	0.24	690.0	2.03E-05	1.02E+01	0.069	0.257	0.501
35	-0.24	718.4	2.20E-05	1.01E+01	-0.069	0.189	0.433
36	0.24	710.0	2.15E-05	1.02E+01	0.069	0.258	0.499
36	-0.24	737.6	2.32E-05	1.01E+01	-0.068	0.189	0.431
37	0.24	730.0	2.28E-05	1.01E+01	0.068	0.258	0.497
37	-0.24	756.9	2.45E-05	1.00E+01	-0.068	0.190	0.429
38	0.24	750.0	2.40E-05	1.01E+01	0.068	0.258	0.495
38	-0.24	776.2	2.57E-05	9.99E+00	-0.068	0.190	0.427
39	0.24	770.0	2.53E-05	1.00E+01	0.068	0.258	0.492
39	-0.24	795.6	2.70E-05	9.94E+00	-0.067	0.191	0.424
40	0.24	790.0	2.66E-05	9.96E+00	0.067	0.258	0.488
40	-0.24	814.9	2.84E-05	9.89E+00	-0.067	0.191	0.421
41	0.24	810.0	2.80E-05	9.91E+00	0.067	0.258	0.484
41	-0.24	834.3	2.97E-05	9.85E+00	-0.067	0.191	0.417
42	0.24	830.0	2.94E-05	9.86E+00	0.067	0.258	0.479
42	-0.24	853.8	3.11E-05	9.80E+00	-0.066	0.192	0.412
43	0.24	850.0	3.08E-05	9.81E+00	0.066	0.258	0.472
43	-0.24	873.2	3.26E-05	9.76E+00	-0.066	0.192	0.406
44	0.24	870.0	3.23E-05	9.76E+00	0.066	0.258	0.465
44	-0.24	892.7	3.40E-05	9.71E+00	-0.066	0.193	0.399
45	0.24	890.0	3.38E-05	9.72E+00	0.066	0.258	0.456
45	-0.24	912.2	3.55E-05	9.67E+00	-0.065	0.193	0.390
46	0.24	910.0	3.54E-05	9.67E+00	0.065	0.258	0.444
46	-0.24	931.7	3.71E-05	9.63E+00	-0.065	0.193	0.379
47	0.24	930.0	3.69E-05	9.63E+00	0.065	0.258	0.429
47	-0.24	951.3	3.86E-05	9.58E+00	-0.065	0.194	0.365
48	0.24	950.0	3.85E-05	9.59E+00	0.065	0.258	0.411
48	-0.24	970.8	4.02E-05	9.54E+00	-0.065	0.194	0.346
49	0.24	970.0	4.02E-05	9.55E+00	0.065	0.258	0.385
49	-0.24	990.4	4.19E-05	9.50E+00	-0.064	0.194	0.321
50	0.24	990.0	4.18E-05	9.50E+00	0.064	0.258	0.258
50	-0.24	1010.0	4.36E-05	9.46E+00	-0.064	0.194	0.194

DRAWDOWN ESTIMATED 0.25 Ft. FROM CENTER OF PUMPING WELL

RECOVERY OR IMAGE WELL NO.	DISCHARGE (gpm)	DIST. FROM O TO WELL (Ft)	u ( )	W(u) ( )	DRAWDOWN INCREMENT (Ft)	CUMULATIVE DRAWDOWN AT END OF LINE OF "I" WELLS (Ft)
0	0.24	0.25	2.57E-12	2.61E+01	0.176	0.538

014768

APPENDIX C

COST ESTIMATE DETAILS

014769

L880

1925

Table C-1

Unit Cost Estimation
Cost of One Sampling Event

Number of Samples Per Event:

	Number
Upgradient Shallow Wells	2
Downgradient Shallow Wells	5
Upgradient Deep Wells	2
Downgradient Deep Wells	3
Field Blanks	1
Upgradient Replicates	1
Downgradient Replicates	1
Surface Water Samples (1 upstream, 2 downstream)	3
Total Number of Samples	18

Cost Per Sample

HSL Analysis	$1,280	
Cost of Analyses Per Event = 18 Samples X $1,280/Sample		$23,000 (Rounded)
Cost of Sampling (Labor, Materials, etc)		10,000
Total Cost Per Event		$33,000

014770

1926

Table C-2

Unit Cost Estimation

Well Installation Cost

Driller's Cost ($35/ft X 60 ft)	2,100
Materials	200
Travel Expense	100

Subtotal Expenses	2,600
+ 10% Handling	260
Labor (Geologist 12 Hrs. @ $50)*	600
Labor (Technician 4 Hrs. @ $35)	140
Labor (Drafting 4 Hrs. @ $40)	160

Subtotal	3,760

Round to:	$4,000

* Includes supervision of driller, well logging, plus 4 hrs. well development.

014771

L561

Table C-3

Unit Cost Estimation

Treatment Unit

	Quantity	Units	Unit Cost	Total Cost
Influent Holding Tank (10,000 gal.)	1	Each	20,000	20,000
Sand Filter (30" diameter)	1	Each	10,000	10,000
Element Filters (15 Micron)	2	Each	2,500	5,000
Feed Pumps (75 gpm)	2	Each	5,000	10,000
GAC Units (200 lbs. ea.)	4	Each	600	2,400
Air Compressor (15 HP)	1	Each	20,000	20,000
Backwash Holding Tank (2,500 gal.)	1	Each	5,000	5,000
Subtotal - Equipment Cost				$ 72,400
Installation (Incl. Foundation, Labor) 40% of Equipment Costs [a]				29,000
Piping & Materials 50% of Equipment Cost [a]				36,200
Electrical Materials & Installation 10% of Equipment Costs [a]				7,200
Instrumentation Materials and Installation 10% of Equipment Costs [a]				7,200
Subtotal Installed Equipment & Matl's Cost				$152,000
Utilities Hook-ups				6,000
			(Rounded)	$158,000

[a] Factors obtained from Peters and Timmerhaus, "Plant Design and Economics for Chemical Engineers", McGraw Hill, 1968.

014772

J194

TABLE C-4
COST ESTIMATION

Well Pumps Installation Cost
(Alternative C - Partial Slurry Wall)

Item	Quantity	Units	Unit Cost	Total Cost
Pumps	3	Each	$ 500	$ 1,500
Control Panel	1	Each	1,100	1,100
Total Equipment Cost				2,600
Installation @40% of Equipment Cost				1,040
Equipment Installed				$ 3,640
Material Cost				
Air Supply Line	2,500	L.F.	$ 2	5,000
Control Air Line	2,500	L.F.	1	2,500
Discharge Piping	2,500	L.F.	7	17,500
Conduit	2,500	L.F.	4	10,000
TOTAL INSTALLATION COST				$ 38,640
			Round to:	$ 39,000

014773

L562

TABLE C-5
COST ESTIMATION

Treatment Unit Operation
Annual Costs

Item	Quantity	Units	Unit Cost	Total Cost
Operating Labor (Half Time)	1,040	Manhours	$ 30	$31,200
Power Consumption (Pumps Only - 2 HP Continuous)	39,000	KWH	0.08	3,120
Carbon Unit Replacement	4	Each	3,600	14,400
Filter Cartridges (1 Per Week Per Filter)	208	Each	50	10,400
Disposal of Silt & Spent Cartidges	24	Drums	200	4,800
Analytical Costs				
1 HSL Test Per Month	12	Each	1,280	15,360
1 Routine Analyses/Wk	52	Each	50	2,600
Capital Maintenance 5% of Capital Cost	1	L.S.	9,150	9,150
Total Operation Cost				$91,030
			Round to:	91,000

014774

J196

TABLE C-6
COST ESTIMATION

Well Pumps Installation Cost
(Alternative D - Recovery Wells)

Item	Quantity	Units	Unit Cost	Total Cost
Pumps	75	Each	$ 500	$ 37,500
Control Panel	25	Each	1,100	27,500
Total Equipment Cost				65,000
Installation @40% of Equipment Cost				26,000
Equipment Installed				$ 91,000
Material Cost				
Air Supply Line	3,250	L.F.	2	6,500
Control Air Line	3,250	L.F.	1	3,250
Discharge Piping	3,250	L.F.	7	22,750
Conduit	3,250	L.F.	4	13,000
				$ 45,500
TOTAL INSTALLATION COST				$ 136,500
			Round to:	$ 137,000

014775

L556

Table C-7

Capital Costs for
Sensitivity Analysis - Volume of Water Treated

Basis of Case	Alternative C Partial Slurry Wall w/ Ground Water Treatment ($M)	Alternative D Recovery Wells w/ Ground Water Treatment ($M)
10 gpm for 25 years	$758	$1,003
10 gpm for 75 years	758	1,003
20 gpm for 25 years	850	1,095
20 gpm for 75 years	850	1,095

[a] Capital costs for different flow rates are calculated from the base case cost estimate by multiplying by the ratio of the flow rates raised to the six-tenths power.

014776

L556

Table C-8

Operating and Maintenance Costs for
Sensitivity Analysis - Volume of Water Treated

Basis of Case	Alternative C Partial Slurry Wall w/ Ground Water Treatment ($M)	Alternative D Recovery Wells w/ Ground Water Treatment ($M)
10 gpm for 25 years	$2,616	$3,136
10 gpm for 75 years	7,848	9,408
20 gpm for 25 years	3,347	4,235
20 gpm for 75 years	10,042	12,704

[a] Assumptions: Fixed costs include ground water monitoring, well pumps maintenance, operating labor, and analytical costs. Variable costs include power consumption, carbon unit replacement, filter cartridges, disposal costs, capital maintenance for the treatment unit, and cost of well pumps operation.

[b] The 75 year operation costs are three times the 25 year costs.

[c] These costs are present value with interest rate cancelling the effects of inflation.